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Development of a Standard Practice for the Design of Durable Open-Graded Friction Course (OGFC) Mixtures with Epoxy Asphalt

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13. Abstract
Open-graded friction course (OGFC) is a thin asphalt mixture surface layer. It is gap-graded with a high percentage of coarse aggregates that are nearly uniform in size, resulting in a high percentage of interconnected air voids and asphalt binder, which provides improved skid resistance, visibility, and decreased pavement-tire noise. However, construction personnel at the Louisiana Department of Transportation and Development (DOTD) reported that conventional OGFC mixtures have durability issues and a shorter service life compared to thin asphalt mixture lifts. The objective of this study was to evaluate the durability and performance of OGFC mixtures containing various types of asphalt binders. Six types of asphalt binders were utilized: unmodified PG 67-22 asphalt binder; conventional styrene-butadiene-styrene (SBS)-modified PG 76-22M asphalt binder; high-SBS content PG 88-28

asphalt binder; diluted epoxy-modified asphalt (EA) binder prepared at two dosage rates (25% and 50% by weight of asphalt binder); and a hybrid PG 76-22G modified asphalt binder prepared with SBS and crumb rubber modifier (CRM). Chemical compatibility and microscopic analyses were first conducted on EA binders to assess the compatibility between multiple asphalt binder sources and the EA binder. The optimal aggregate structures were determined based on the minimum required air voids and voids in coarse aggregate. Rheological and chemical characterization was conducted on the selected asphalt binders. Further, a suite of physical and mechanical mixture tests was performed to assess the performance of OGFC mixtures, including: draindown for mixture; draindown during production, storage, and construction; permeability for water drainability; loaded wheel track (LWT) for rutting and moisture susceptibility; Cantabro abrasion loss for durability; and indirect tensile strength for moisture susceptibility. Results from SARA fractions and Fourier Transform Infrared Spectroscopy tests showed that EA binders had the highest aging resistance. OGFC mixtures with EA binders exhibited the lowest draindown values. Mixtures with 50%EA and PG 88-28 binders demonstrated the highest resistance to rutting and moisture damage, as measured by LWT test for unconditioned and conditioned samples, respectively. All mixtures studied were durable, as determined by Cantabro abrasion loss values of 20% and 30% on unaged and aged (5 and 15 days) samples, respectively. Mixtures with 25%EA and 50%EA binders exhibited the highest resistance to moisture damage, as indicated by their tensile strength ratio results. Cost-effectiveness ratio (CER) results showed that mixtures containing 25%EA and 50%EA binders have higher effectiveness compared to the conventional OGFC mixture with PG 76-22M when tested for 30 days aged moisture-conditioned Cantabro specimens.

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Acknowledgments

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Implementation Statement

It is recommended to revise Section 501 *Thin Asphalt Concrete Applications* of the Louisiana DOTD specifications for Road and Bridges based on findings of this research. Specifically, Table 501-1 *Asphalt Mix Design Requirements* is revised for OGFC mixtures as follows:

Table 501-1
Asphalt Mix Design Requirements

Mix Type	Current OGFC	Revised OGFC
Asphalt Cement Grade	PG 76-22m	50%EA or PG 88-28HM
Stone-on-stone contact, ASTM D 7064	n/a	$VCA_{mix} \leq VCA_{drc}$
Gyratory Revolutions[1]	50	50
Minimum AC content, %	6.5	6.5
Air Voids, %[2]	18-24[3]	18-24[3]
Sands, Max. %	0	0
RAP, Max %	0	0
LWT rut depth, 12 mm (max) @ no. passes, AASHTO T 324[4]	5000	5000
Water Susceptibility, LWT rut depth, 12 mm (max) @ no. passes, freeze-thaw conditioned samples, AASHTO T 283	n/a	5000
Draindown, % max[5]	0.3	0.3
Water Susceptibility, Boil Test, DOTD TR 317, % min	90	n/a
Durability, Cantabro Abrasion Loss, ASTM D 7064, % max, compacted specimens aged for 5 days at 85°C	n/a	30[6]
Min. Tack Coat Application Rate, Undiluted gal/sq.yd. (0.40 gal/sq.yd maximum)[7]	0.15	0.15

Notes: na: not available; VCA: voids in coarse aggregate; drc: dry-rodded voids in coarse aggregate; 50%EA: asphalt binder PG 67-22 containing 50% epoxy by weight of binder; PG 88-28HM: high-SBS modified asphalt binder.

$$VCA_{drc} = \frac{G_{CA} * \gamma_w - \gamma_s}{G_{CA} * \gamma_w}$$

$$VCA_{mix} = 100 - \left[\frac{G_{mb}}{G_{CA}} * P_{CA}\right]$$

where,

γ_w= unit weight of water;

γ_s= bulk density of the coarse aggregate fraction in the dry-rodded condition;

G_{CA} = bulk specific gravity of the coarse aggregate;

G_{mb} = bulk specific gravity of compacted mixture; and

P_{CA}= percentage of the coarse aggregate in the total mixture.

[1]Compact specimen according to AASHTO T 312.

[2]Design target voids at mid-point of void requirement. Full range allowed for OGFC.

[3]As computed using the measure of the physical volume (weight of compacted specimen)/(height of compacted specimen x area of the compacted specimen).

[4]Compact LWT specimen to the target voids.

[5]As measured in accordance with ASTM D 6390.

[6]As measured in accordance with ASTM D 7064 for compacted specimens aged for 5 days at 85°C.

[7]See 501.02.1 for allowable tack coats.

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Introduction

Open-graded friction course (OGFC) is a thin asphalt mixture surface layer. It is gap-graded with a high percentage of coarse aggregates, with a minimum of fines that are nearly uniform in size, resulting in a higher percentage of interconnected air voids and asphalt binder [1, 2, 3]. OGFC is designed primarily as a thin-wearing surface with high permeability that allows for the lateral drainage of rainwater [4, 5, 6]. In general, OGFC surfaces reduce hydroplaning, reduce splash-and-spray, and enhance roadway visibility and skid resistance under wet weather conditions [7, 8]. Additionally, OGFC surfaces improve pavement smoothness and reduce pavement-tire noise. However, conventional OGFC mixtures have been found to have durability issues and raveling distress, which causes rough pavement surfaces, less ride quality, and more traffic noise [9, 10, 11].

One solution to overcome the durability issues of OGFC mixtures is to consider additives such as fibers and styrene-butadiene-styrene (SBS) asphalt binder modifiers [12, 13, 14]. The addition of SBS modifier to OGFC mixtures has been shown to improve performance in terms of draindown, rutting resistance, raveling resistance, and moisture susceptibility [15, 16, 17]. Additionally, SBS modifier has been found to have a positive effect on asphalt binder, such as increasing its elasticity and stiffness [17, 18], which results in improved mixture stiffness. Similarly, other studies investigated the effect of high SBS content (greater than 6%, by weight of asphalt binder) on the performance of OGFC [19, 20, 21, 22]. For example, Lin et al. (2019) showed that a high percentage of SBS (9%) can improve the rheological properties of asphalt binders, resulting in improved rutting and cracking resistance [21]. Moreover, Xu et al. (2016) conducted a study which revealed that incorporating a high percentage of SBS in porous mixtures resulted in improved resistance to raveling when compared to conventional SBS mixtures [22]. Further, the performance of an OGFC mixture can also be enhanced by incorporating crumb rubber modifier (CRM), a sustainable recycled material obtained from waste tires, into the asphalt binder. Combining crumb rubber (CR)-modified binder with SBS improves the performance of asphalt pavements. Researchers have investigated the use of crumb rubber and SBS in modifying asphalt mixtures. Howard et al. compared the effectiveness of three asphalt binder modifiers in improving mixture performance: CRM, SBS, and hybrid SBS/CRM [23]. The aforementioned study found that hybrid binder containing 1–2% SBS and 3–8% CRM by weight of asphalt binder improved rutting and cracking resistance.

Epoxy asphalt (EA) binder has also been used in OGFC mixtures [24]. EA binder is a two-phase chemical reaction in which the continuous phase is a thermosetting resin, and the discontinuous phase is a base asphalt binder and curing agent [25]. The curing agent is blended with the base asphalt binder to form Part B (base asphalt binder and curing agent), and the resin (commonly referred as Part A) is blended with Part B and left for curing to form a thermosetting EA binder. After curing, EA binder behaves as a thermosetting material that will not melt after being fully cured (i.e., an irreversible chemical reaction) [26, 27]. After complete curing, EA binder is tougher, more elastic, and does not soften at high temperatures compared to conventional asphalt binders [5]. EA binder has gained much attention recently because of its successful application in pavements [28], and studies show that EA binder could provide a promising solution for the durability issues associated with flexible pavement's premature failure, especially due to heavy traffic volume and extreme weather conditions [29, 30]. Xu et al. reported advantages in the use of EA binder, such as excellent adhesion, thermal stability, and less aging susceptibility [27]. Further, Wu et al. reported that OGFC modified with EA binder exhibited extended service life and extended durability and fatigue resistance compared to other OGFC mixtures containing 4% SBS polymer-modified asphalt binder [26]. The study also evaluated asphalt mixtures containing asphalt binder modified with EA and SBS. The study revealed that the asphalt mixtures with EA binder were more durable than the asphalt mixtures modified with 4% SBS asphalt binder in terms of abrasion loss and fatigue life [26]. Another study by Luo et al. showed that EA binder is tough, elastic, and has good resistance to cracking, rutting, and fatigue [24].

Conventional OGFC mixtures have been found to exhibit durability issues and raveling distress, leading to rough pavement surfaces with poor ride quality and increased traffic noise. It is necessary to find feasible solutions to overcome these issues and extend the service life of OGFC mixtures. This research presents the results of physical and mechanical tests of OGFC mixtures containing different types of asphalt binders to improve their mechanical properties and durability. Additionally, a cost-effectiveness analysis was performed at the optimum asphalt binder content to ascertain the cost-effectiveness of the different techniques for enhancing the durability of OGFC mixtures.

Literature Review

This section provides a comprehensive literature review regarding open-graded friction course (OGFC) mixture's design, advantages, and current shortcomings. OGFC mixtures are a specialized type of asphalt pavement designed primarily as a thin-wearing surface with high air voids for many benefits. Although OGFC mixtures are not designed to add structural capacity to the road, they offer numerous functional benefits. These benefits are divided into two primary categories: those that benefit road users, and environmental benefits vital to many transportation officials.

Background

OGFC originated in California in the 1940s and subsequently expanded to other states. In 1988, 27 states were using OGFC; however, this number declined to 22 in the late 1990s. The decline in OGFC usage was attributed to the premature failure of OGFC mixtures, notably the failure caused by raveling [31]. In Louisiana, OGFC mixture usage started in the late 1960s, as the Louisiana Department of Transportation and Development (DOTD) required the use of OGFC on all roads with average daily traffic greater than 4,000 vehicles. However, similar to other states, premature failures of these mixtures were observed during the late 1980s, resulting in a moratorium on their usage. The issues encountered with the failed OGFC mixtures were related primarily to durability and moisture damage, which manifested in raveling and stripping distresses. To address these issues, several changes were made to the specifications, such as limiting the maximum moisture content for the aggregate and increasing the minimum ambient air temperature. These changes improved the performance of OGFC mixtures, and as a result, the moratorium was lifted [32, 33].

Aggregate Selection

The design of OGFC mixtures begins with selecting an aggregate structure that assures a minimum air void content requirement of 18%. To achieve a stone-on-stone contact aggregate skeleton, the voids in the coarse aggregate for the mixture (VCA_{mix}) should be less than the voids in the aggregate for the uncompacted coarse aggregate (VCA_{drc}). Aggregates heavily influence the performance of asphalt mixtures due to their properties and structure, as they constitute the largest proportion within a typical asphalt mixture. Therefore, aggregate quantity and quality are crucial for optimal performance. For example, the gradation is carefully designed to achieve a high air void content that facilitates rainfall drainage. The nominal maximum aggregate size (NMAS), as well as individual sieve sizes, have been found to be key factors in determining the general performance of OGFC mixtures [34, 35, 36, 37, 38]. Some researchers have reported that increasing the percent passing sieve #4 of OGFC mixtures improves durability, as measured by a reduction in Cantabro abrasion loss values [34, 35]. Similarly, Nekkanti et al. showed that using NMAS of 9.5 mm instead of 12.5 mm reduced Cantabro abrasion loss by 55%. Further, they found that increasing the percent passing sieve #4 (above 20%) decreased Cantabro abrasion loss by 55% [35]. According to the NCHRP 01-55 study, the percent passing sieve #200 was also found to increase durability, as measured by a decrease in Cantabro abrasion loss values [3]. In short, using aggregate gradation that has a smaller NMAS and higher percent passing sieves #4 and #200 resulted in the enhanced durability of OGFC mixtures. The aforementioned properties fall under the effect of aggregates on OGFC in terms of quantity. In contrast, the quality of aggregates plays a major role in determining the performance of OGFC mixtures. For instance, the aggregates should be angular, rough, and clear of contaminants and deleterious materials to ensure stone-on-stone contact, which enhances interlock and increases stability. Further, the aggregates should be hard, abrasion-resistant, and sound, so as not to degrade under freeze-thaw conditions or chemical weathering. Additionally, aggregates with low absorption and high compatibility with asphalt binder are typically used to ensure adequate coating and adhesion, which enhances long-term performance and durability [1, 4, 5, 11, 19].

Asphalt Binder Selection

Various modifiers, such as polymers, CR particles, and epoxy modifiers, can be utilized to improve the mechanical and rheological properties of asphalt binders used for OGFC production. For example, SBS improves asphalt binder elasticity, resistance to aging and temperature fluctuations, and stiffness. Additionally, it improves asphalt mixture rutting and cracking resistance [39, 40, 41, 42]. Similarly, crumb rubber has been found to be effective in improving asphalt binder's flexibility, elasticity, and recovery properties, enhancing fatigue cracking resistance and providing noise reduction benefits because of its excellent damping ability [43, 44, 45, 46, 47, 48]. Further, EA binder has been shown to improve the adhesion between asphalt binder and aggregate particles, which reduces the mixture's susceptibility to moisture damage or stripping. Additionally, EA binder tends to exhibit high resistance to rutting, thermal and fatigue cracking, and oxidative aging [1, 5, 24, 25, 28]. Therefore, modified asphalt binders are commonly used in highway construction to improve functionality and durability and reduce maintenance needs.

OGFC Mix Design

OGFC mix design is a critical process that involves selecting the optimum combination of aggregates, asphalt binder, and additives to achieve the desired performance in the field. The mix design begins with the selection of the optimum aggregate gradation and asphalt binder content, followed by the mixture evaluation. Two standard specification methods are typically used for OGFC mix design: ASTM D7064, "Standard Practice for Open-Graded Friction Course (OGFC) Mix Design" [49], and AASHTO PP 77, "Standard Practice for Materials Selection and Mixture Design of Permeable Friction Courses (PFCs)" [50]. Nonetheless, many state DOTs design OGFC mixes following their own standards. For example, Louisiana DOTD designs its OGFC mixes following Section 501 of the 2016 "Louisiana Standard Specifications for Roads and Bridges" [51].

When selecting aggregate gradation, it is important to ensure that it satisfies the air void content requirement. The optimum asphalt binder has upper and lower limits. The upper limit is determined by the draindown test, which ensures acceptable draindown limits during mixture design and/or field production. The lower limit, on the other hand, is determined according to the Cantabro abrasion loss test to ensure durability and long-term performance. Finally, the selected mix design is evaluated for draindown, permeability, permanent deformation, moisture damage, and durability using the draindown test, permeability test, load wheel track (LWT) test, Modified Lottman (TSR) test, and Cantabro abrasion loss test, respectively.

Summary of Previous Research Studies

The following findings were made after a thorough review of literature regarding laboratory investigations and insights gained from the construction of OGFC mixtures [3, 31, 34, 52, 53].

- Through previous studies, researchers have firmly established an inverse relationship between combined aggregate bulk specific gravity and minimum asphalt content. For example, a high aggregate bulk specific gravity (approximately 2.9) allows for the use of low asphalt content (approximately 5.5%); however, a lower aggregate bulk specific gravity (approximately 2.4) requires an increased optimum asphalt content of 6.8% to ensure satisfactory performance [31, 34]. Thus, the density of aggregates significantly influences the selection of minimum asphalt content for the acceptable performance of

OGFC mixtures in the field, with higher-density aggregates potentially reducing costs due to the use of lower asphalt content.

- The mix design should ensure the functionality and durability of OGFC mixtures. Functionality is primarily measured by permeability, which ensures rapid surface water removal. The optimal asphalt content should provide a balance between draindown and durability, as measured by draindown and Cantabro abrasion loss tests, respectively. The lower asphalt content results in lower costs and satisfies draindown requirements. However, a mix design that satisfies draindown requirements may not necessarily meet durability requirements. Therefore, it is of vital importance to establish boundary limits for the asphalt content, with a minimum limit to ensure enough durability and a higher limit to ensure acceptable draindown.
- The gradation of the aggregates used for the OGFC mixture influences the asphalt binder content. Aggregates with coarse gradation, characterized by low surface area, require less asphalt binder to coat the aggregates, whereas aggregates with fine gradations have higher surface areas and require more asphalt binder to coat the aggregates. The gradation type is governed by the break-point sieve determination, which is defined as the smallest sieve to retain 10% or more. The break-point sieve can also be visually defined as the sieve size that has a definite break in the slope of the aggregate gradation curve. A higher percent passing value for the break-point sieve results in fine gradation, whereas a lower percent passing value for the break-point sieve results in coarse gradation. Therefore, it is important to closely control the percent passing the break-point sieve.
- Previously, premature failures of OGFC mixtures were attributed to the absence of fibers and the use of unmodified asphalt binder. Fibers enhance stability and ensure acceptable draindown levels, while modified asphalt binder maintains stone-on-stone contact and keeps aggregates in place. To prevent premature failures and extend service life, it is crucial to incorporate modified asphalt binder and fibers in the construction of OGFC mixtures.
- The LWT test is better for characterizing the moisture susceptibility of OGFC mixtures compared to the Modified Lottman test, as stated by Tsai et al. in a study conducted at the Pavement Research Center at the University of California, Davis [53]. The same study suggested that the 50 gyrations used for designing OGFC mixtures, as recommended by NCAT, appear insufficient to generate the same compaction effort to attain the typical aggregate interlock observed in field conditions. Accordingly, it was recommended that the number of mix design gyrations be increased to 70 to enhance durability and rutting resistance.

Shortcomings and Research Gaps

Using OGFC mixes on highways improves road safety and increases road users' satisfaction. However, several issues may arise from using OGFC mixes, and these need to be addressed to enhance their performance and functionality. For example, the open structure of OGFC mixes can be susceptible to clogging with fines and other debris, which inhibits drainage and adversely affects skid resistance. Further, premature failure in the form of raveling and stripping distress remains common, further hindering the widespread use of OGFC mixes on highways. Finally, durability, shorter service life, and frequent maintenance are still primary concerns for using OGFC mixes. Therefore, this study investigates the use of unconventional techniques, including the use of epoxy-modified asphalt (EA) binders and polymer-modified asphalt binders containing high SBS content, to improve the durability of OGFC mixes.

Objective

The objective of this study was to evaluate the durability and performance of OGFC mixtures containing various types of asphalt binder.

Specific objectives included the following:

- Determine if EA binder can significantly improve the durability and performance of OGFC mixtures at multiple dilution rates;
- Compare the effect of various modifiers (SBS, SBS/CRM, and EA) on asphalt binders' rheology and OGFC mixture performance;
- Determine the effect of different asphalt binder contents on the physical and mechanical performance of OGFC mixtures; and
- Ascertain the cost-effectiveness of various asphalt binder types.

Scope

A 12.5 mm OGFC mixture was designed following ASTM D 7064, "Standard Practice for Open-Graded Friction Course (OGFC) Mix Design" [49]. The aggregate structures were optimized based on the required minimum air void content and voids in coarse aggregates. Two asphalt binder contents (P_b) were considered: 6.5% and 7% by weight of asphalt binder. An SBS-modified PG 76-22M binder at a P_b of 6.5% was selected as a baseline for the selection of the optimum aggregate structure, while a P_b of 7% for the PG 76-22M binder was used to ascertain the effect of increased asphalt binder content on the durability of OGFC mixtures. Six types of asphalt binders were utilized: unmodified PG 67-22 asphalt binder; conventional styrene-butadiene-styrene (SBS)-modified PG 76-22M asphalt binder; high-SBS content PG 88-28 asphalt binder; diluted epoxy-modified asphalt (EA) binder prepared at two dosage rates (25% and 50% by weight of asphalt binder); and a hybrid PG 76-22G modified asphalt binder prepared with SBS and crumb rubber modifier (CRM). The compatibility between multiple asphalt binder sources and the EA binder was first determined, followed by a suite of rheological and chemical tests to evaluate the modified asphalt binders: performance grading (PG); multiple stress creep recovery (MSCR); frequency sweep (FS); linear amplitude sweep (LAS); Fourier-transform infrared (FTIR) spectroscopy; and SARA analysis. Further, a suite of physical and mechanical tests was conducted to assess OGFC mixtures, including: draindown test; permeability test; LWT test for rutting; LWT and Modified Lottman tests for moisture susceptibility; and Cantabro abrasion loss test for durability. Triplicate samples were tested, except for LWT, where four specimens were used. Finally, a cost-effectiveness analysis was conducted for the evaluated asphalt binder types.

Methodology

Materials

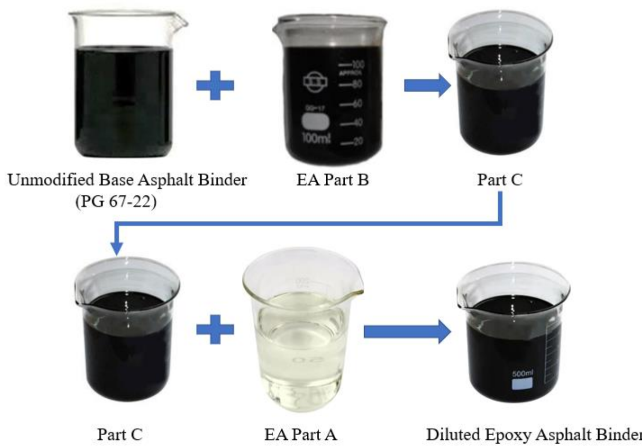
This section describes the component materials used in this study. Five types of asphalt binder were considered in the study: conventional styrene-butadiene-styrene (SBS)-modified PG 76-22M asphalt binder; high-SBS content PG 88-28 asphalt binder; diluted epoxy-modified asphalt (EA) binder prepared at two dosage rates (25% and 50% by weight of asphalt binder); and a hybrid PG 76-22G modified asphalt binder prepared with SBS and crumb rubber modifier (CRM). Additionally, three sources of commonly used aggregates in Louisiana (#78 limestone, #78 sandstone, and #11 limestone) were used in the study. Cellulose fibers were used in the OGFC to prevent draindown. All materials and OGFC mixture designs met the requirements specified in Sections 501, 1002, and 1003 of the 2016 "Louisiana Standard Specifications for Roads and Bridges" [51].

Epoxy Asphalt (EA) Binder

The undiluted commercial EA binder used in this study contains two components: Part A (epoxy resins) and Part B (curing agents mixed with base asphalt binder). According to the manufacturer's recommendation, Parts A and B are to be blended at a certain fixed stoichiometric weight ratio (Part A = 19.4%, Part B = 80.6%). In this study, the blend of Parts A and B was further diluted with unmodified PG 67-22 base asphalt binder at two dosages (25%EA and 50%EA), producing a diluted EA binder. A blend of 25% (by weight) of EA (Parts A and B) and 75% (by weight) of unmodified PG 67-22 base asphalt binder is hereafter referred to as 25%EA. A similar designation is followed for 50%EA. For illustration, 100g of 25%EA contains 4.9g (Part A: 100*19.4%*25%) of resin and 95.1g (Part B: 100-4.9) of curing agent and unmodified base asphalt binders, the latter of which can be pre-blended for convenience.

Figure 1 shows the process of producing diluted EA binder. The unmodified PG 67-22 base asphalt binder and Part B were first heated to 121ºC and 155ºC, respectively, and blended for two minutes to produce Part C. Next, Part C and Part A were placed in the oven at 121ºC for 20 min. and further blended for 30 sec. to form a diluted EA binder. It should be noted that both 25%EA and 50%EA binders have a thermoplastic behavior because of the low resin content (approximately 4.9% and 9.8%, respectively). However, as the diluted EA binder approaches 100%EA, the EA binder converts to a thermoset material that does not melt after being fully cured.

Figure 1. Epoxy asphalt (EA) binder producing process



EA/Base Asphalt Binder Source Chemical Compatibility Based on Soxhlet Extraction Method

A chemical compatibility experiment was performed to assess the colloidal stability and chemical compatibility between the EA components and three unmodified base asphalt binder sources commonly used in Louisiana, according to ASTM D 7173, "Standard Practice for Determining the Separation Tendency of Polymer from Polymer-Modified Asphalt" [54] and ASTM C 613, "Standard Test Method for Constituent Content of Composite Prepreg by Soxhlet Extraction" [55]. Three PG 67-22 base asphalt binders commonly used in Louisiana were evaluated. First, a cigar tube test was performed by pouring 62 g of diluted EA binder into aluminum tubes and conditioning them at 121ºC for 8 hrs. in a vertical position. Next, the aluminum tubes were removed and placed in a freezer at -10ºC (± 10ºC) for 4 hrs. Afterward, the aluminum tubes were cut into approximately three equal parts (top, middle, and bottom); see Figure 2.

Figure 2. Storage stability: cigar tube at conditioning, aluminum tube cutting, and retrieving EA binder from aluminum tubes

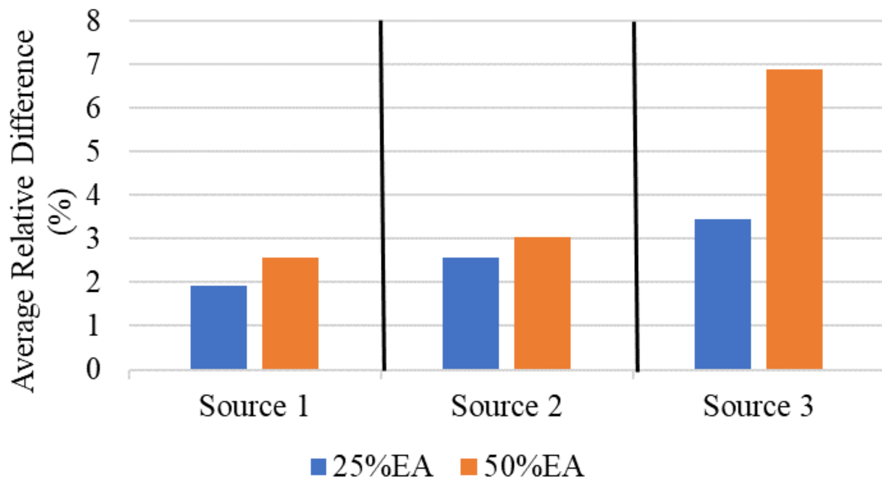
The top and bottom parts were then placed in small containers (100 ± 20 mL) and conditioned in an oven at 121ºC until the asphalt was sufficiently fluid to remove the aluminum tube. Afterward, the top and bottom parts of the diluted EA binder were subjected to Soxhlet asphalt extraction [55]. Figure 3 shows the Soxhlet extraction process, in which 9 g of diluted EA binder were poured into a thimble and the asphaltic part was washed away from the resin using trichloroethylene (TCE) solvent, at a rate of 3–10 reflux changes per hr. Soxhlet extraction was stopped after 20 reflux changes or 4 hrs., whichever came first. The thimbles were placed under a hood overnight, then weighed. The average relative difference between the top and bottom parts of the samples was used as an indicator of chemical compatibility between the unmodified base asphalt binder and the EA binder.

Figure 3. Soxhlet asphalt extraction: thimble at the beginning and end of the test

Figure 4 presents the compatibility test results for the three unmodified base asphalt binder sources evaluated. Unmodified base asphalt binder Source 1 provided the lowest percent difference (i.e., most compatible) between the tube’s top and bottom portions for both

25%EA and 50%EA binders. Thus, it is the most compatible choice to dilute the EA binder used in this study.

Figure 4. Compatibility test result through Soxhlet asphalt extraction



EA/Base Asphalt Binder Source Chemical Compatibility Based on Confocal Laser-Scanning Microscopy

Confocal laser-scanning microscopy (CLSM) was used to further evaluate the chemical compatibility of the EA binder with the unmodified base asphalt binder sources considered. EA binder particles are detected in the image when illuminated with a point laser source of a certain wavelength that causes fluorescence. A Lecia TSC SP8 microscope was irradiated by objectives 40 and 63 (both at zoom x1) with 488 nm wavelength light, and the fluorescence was observed in the range of 500–550 nm wavelengths. All images were captured in two dimensions, in 1,024×1,024-bit TIFF format. Diluted EA binder samples were prepared and cured for 4 hrs., stirred thoroughly, and poured on a glass slide. The glass slide was placed on a hot plate at 120ºC for 5 min., and the drop was covered by a cover slip in order for the small drop to uniformly cover the entire area of the slip. The cover slides were then allowed to cool down to room temperature prior to testing [56], as shown in Figure 5.

Figure 5. Microscope glass slide, and Lecia TSC SP8 microscope

Figure 6 presents the particle size results of the CLSM images analyzed using ImageJ software [57]. First, each image was converted to 8-bit format, adjusted for contrast threshold, and auto-edge detected. Next, the area of each EA binder particle was calculated and compared to other images, based on the criteria of choosing the image exhibiting the least value of mean particle size, median particle size, and standard deviation [1].

Figure 6. CLSM analysis procedure

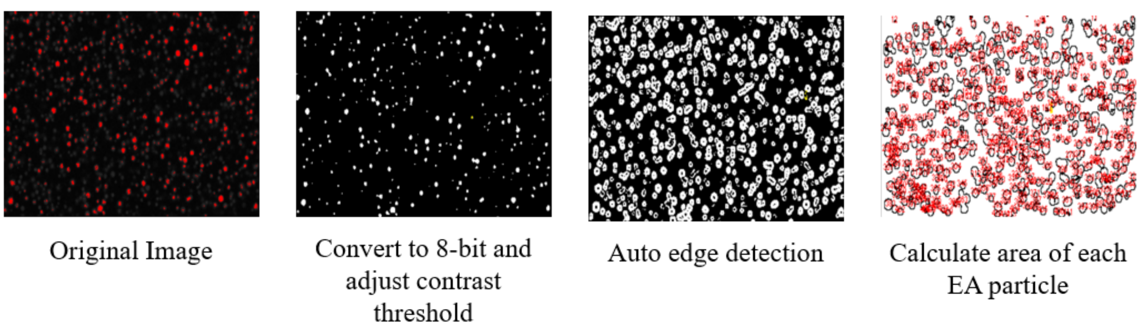
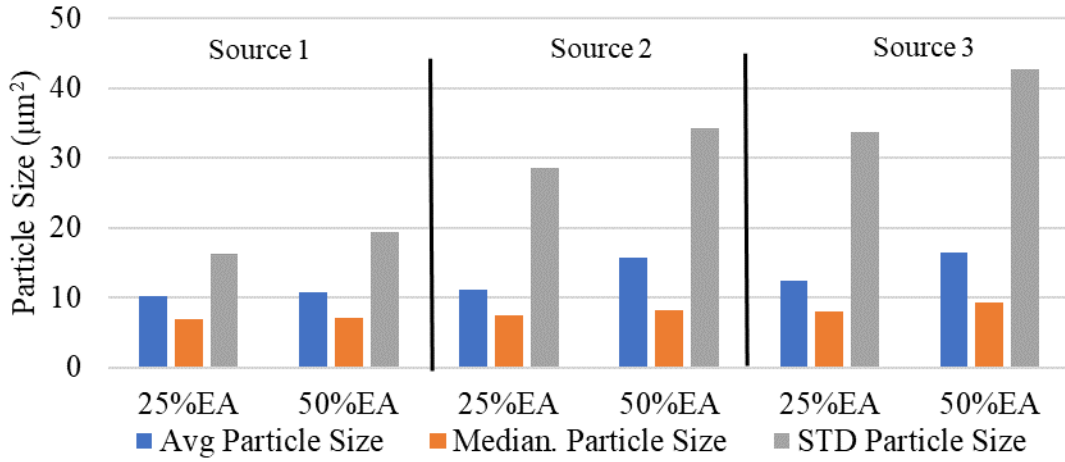


Figure 7 presents a summary of particle size results for three base binder sources using ImageJ software [57, 58]. Similar to the chemical compatibility results, Source 1 unmodified base asphalt binder had the least mean/median particle sizes as well as the least standard deviation, which suggests its highest compatibility with EA compared to other binder sources. Since binder Source 1 provided the best results in the two compatibility tests, it was selected to blend with the EA binder to provide the diluted EA binder used in the preparation of OGFC mixtures.

Figure 7. CLSM test results



SBS and CRM Modified Asphalt Binders

All asphalt binders were graded according to AASHTO M 320, "Standard Specification for Performance-Graded Asphalt Binder" [59] and "Louisiana Specifications for Roads and Bridges" [51]. The conventional asphalt binder is SBS polymer-modified and meets Louisiana specifications for PG 76-22M [51]. The PG 88-28 asphalt binder contained approximately 7% SBS, whereas PG 76-22G is a hybrid binder formulated with approximately 2% SBS and 30 mesh crumb rubber modifier (CRM). The 25%EA and 50%EA binders contained PG 67-22 unmodified base asphalt binder diluted with 25%EA and 50%EA, respectively. Table 1 shows the basic properties of the studied asphalt binders.

Table 1. Asphalt binder basic properties

Asphalt Binder ID	Modifier Dosage	Asphalt Binder Grade
76-22M	3.5% SBS	PG 76-22
76-22G	1.5% SBS / 6.5% CRM	PG 76-22
88-28	7.5% SBS	PG 88-28
25%EA	4.9% Resin 95.1% Base binder and curing agent	PG 70-22
50%EA	9.8% Resin 90.2 Base binder and curing agent	PG 70-22

SBS: Styrene-butadiene-styrene; CRM: Crumb Rubber; EA: Epoxy Asphalt.

Asphalt Binder Laboratory Tests

Superpave Performance Grading

Superpave performance grading was performed using the dynamic shear rheometer (DSR) on all asphalt binders to evaluate their high-temperature grading, intermediate-temperature grading, and low-temperature grading, following AASHTO M 320, “Standard Specification for Performance-Graded Asphalt.” The rotational viscosity (RV) was conducted at 135°C, according to AASHTO T 316, “Standard Method of Test for Viscosity Determination of Asphalt Binder Using Rotational Viscometer” [60]. The short- and long-term aging were simulated following the standard aging procedures for the rolling thin-film oven (RTFO) test, according to AASHTO T 240 [61], and pressurized aging vessel (PAV), according to AASHTO R 28 [62]. Moreover, the critical temperature difference (ΔT_c) parameter can be determined from the Superpave performance grading using the bending beam rheometer (BBR) test [63], according to Equation 1.

$$\Delta T_c = T_S - T_m \qquad (1)$$

where,
T_s = the critical temperature at which the flexural stiffness (S) of the beam equals 300 MPa; and
T_m = the critical temperature at which the slope (m) of stiffness versus time in the scale equals 0.300.

Note that both critical temperatures were evaluated at a creep loading time of 60 sec.

The ΔT_c parameter gives insight into the relaxation ability of an asphalt binder, which contributes to non-loading distresses. In general, a less negative ΔT_c suggests higher resistance to cracking.

Multiple Stress Creep Recovery (MSCR) Test

The MSCR test was performed to evaluate the high-temperature rutting resistance of the asphalt binders. As rutting performance is a short-term concern, the RTFO-aged asphalt binder was utilized. This test was conducted at a high PG temperature using DSR, in accordance with AASHTO T 350 [64]. The parallel plate geometry with a 25 mm diameter and a 1 mm gap was used, and the test was conducted at 67°C. This test starts with 20 cycles, each consisting of 1-sec. creep loading with a low shear stress of 0.1 kPa and 9-sec. zero stress recovery, followed immediately with the same 10 creep-recovery cycles, except that the creep load is increased to 3.2 kPa. A number of parameters can be determined from the MSCR test, including percentage recovery and non-recoverable creep compliance (J_{nr}) for each of the two stress levels of 0.1 kPa and 3.2 kPa. It has been found that J_{nr} evaluated at the higher stress level, denoted as $J_{nr3.2}$, is adequately correlated with the rutting performance of asphalt mixtures in the laboratory, as well as in the field. In this study, $J_{nr3.2}$ and recovery% at 3.2 KPa stress level will be employed as the parameter to evaluate the effect of different modified asphalt binders on rutting performance.

Frequency Sweep (FS) Test

The frequency sweep (FS) test was performed to characterize the viscoelastic properties of asphalt binders at multiple temperatures and frequencies, according to ASTM D 7175 [65]. A parallel plate geometry with an 8 mm diameter and a 2 mm gap was used. The test temperatures were 15°C, 30°C, and 45°C. For each temperature, the test was conducted at various frequencies ranging from 0.1 to 100 rad/sec. The strain level was controlled at 1% to ensure the asphalt binder was in the linear viscoelastic region, and the data was used to construct the master curve for dynamic shear modulus and phase angle. Isotherms of dynamic modulus were shifted with respect to the selected reference temperature. The Christensen Anderson model was used to fit a function on the isotherms; see Equations 2-4 [66].

$$|G^*| = G_g\left[1 + \frac{\omega_c}{\omega_r}^{\frac{log2}{R}}\right]^{\frac{-R}{log2}} \quad (2)$$

$$\omega_r = \omega \,.\, a_T \quad (3)$$

$$\log(a_T) = a_1(T - T_R)^2 + a_2(T - T_R)^2 \quad (4)$$

where,
$|G^*|$= dynamic shear modulus;
G_g= glass modulus;
ω_c= crossover angular frequency;
ω_r= reduced angular frequency;
R= rheological index;
ω= measured angular frequency;
a_T= shift factor;
T_R= reference temperature;

a_1 and a_2 = regression coefficients for the shift factor function; and
δ= phase angle.

The Glover-Rowe (G-R) parameter was calculated according to Equation 5. This parameter is a measure for the stiffness and elasticity properties of long-term aged asphalt binder. A lower value is desired, as it indicates better resistance to cracking (i.e., more ductile and less brittle). The initiation of cracking is expected to occur as the G-R value approaches 180 kPa, and substantial cracking is expected to occur as the G-R value reaches 480 kPa [67, 68, 69].

$$G - R\ Parameter = \frac{|G^*| \times (\cos\delta)^2}{\sin\delta} \tag{5}$$

Linear Amplitude Sweep (LAS) Test

The LAS test was performed according to AASHTO TP 101 to evaluate the fatigue characteristics of the studied asphalt binders [70]. The test was conducted at 18°C using parallel plate geometry with an 8 mm diameter and a 2 mm gap. A frequency sweep test was first performed, followed by an amplitude sweep step. The frequency sweep ranged from 0.1 to 100 rad/sec. at 0.1% strain amplitude, and the amplitude sweep was performed at a fixed frequency of 10 Hz by applying a torsional strain, which increased linearly from 0.1% to 30%. The LAS test result was analyzed based on viscoelastic continuum damage (VECD) theory to obtain a damage characteristic relationship. The fatigue parameter A_{LAS} was calculated according to Equation 6. Higher A_{LAS} values are desired, as they represent higher crack resistance [66].

$$A_{LAS} = \left[\frac{1}{2E_R} C_1 C_2 . (|G^*_{LVE}|)^2\right]^{-\alpha} . f(kQ)^{-1} . \left(S_f\right)^k \tag{6}$$

$$Q = \int_0^{2\pi/\omega_r} (\sin(\omega_r \zeta))^{2\alpha} d\zeta \tag{7}$$

where,
E_R= reference modulus;
C_1 and C_2= regression coefficients;
$|G^*_{LVE}|$= linear viscoelastic modulus;
f= loading frequency (10 Hz);
k= 1+(1- C_2) α;
S= internal state variable for damage intensity;
S_f= S-value at failure;
ω_r= reduced angular frequency;

α= damage evolution rate; and

Q= loading condition (temperature and frequency) factor.

SARA Fractions Test

SARA analysis determines the chemical composition of asphalt binder by fractionating it into saturates, aromatics, resins, and asphaltenes. Asphaltenes are the pentane- or heptane-insoluble component of asphalt binder, while maltenes are the soluble component that can be further separated into the other three fractions (saturates, aromatics, and resins). Asphaltenes consist of extremely complex, highly polar molecules; they exhibit a very high tendency to associate into molecular clusters and play a significant role as viscosity builders in the rheology of asphalt binder [71]. During the oxidative aging process, ketones are formed, which significantly changes the polarity and solubility of the associated aromatic components, leading to their agglomeration to form the asphaltene component. The resulting increase in the asphaltene fraction then becomes the primary reason for the increase in asphalt viscosity due to aging [71].

The colloidal instability index (CII) can be obtained as the ratio of the sum of the saturates and asphaltene contents to that of the resins and aromatic contents; see Equation 8. A low colloidal instability index indicates a well-dispersed, homogeneous system, while a high colloidal instability index suggests a more gel-like system that is less dispersed and more heterogeneous. Therefore, asphalt binders with low colloidal instability indices are expected to exhibit better resistance to cracking. The asphaltene component was first separated from the maltenes in accordance with ASTM D 3279 [72]. Next, the maltenes component was fractionated on an Iatroscan TH-10 Hydrocarbon Analyzer to obtain the components of saturates, aromatics, and resins. The n-pentane was used to elute the saturates, and a 90/10 toluene/chloroform mixture was used to elute the aromatics. The resins were not eluted and remained at the origin.

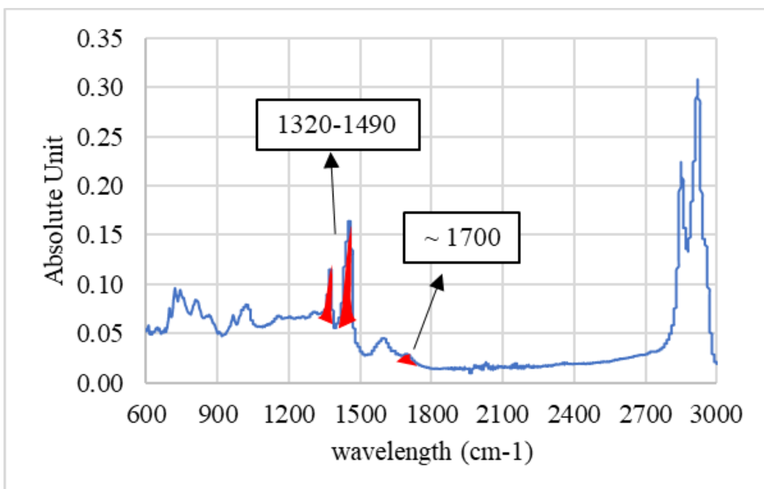
$$CII = \frac{\text{Saturates + Asphaltenes}}{\text{Resins+ Aromatics}} \tag{8}$$

Fourier-Transform Infrared (FTIR) Spectroscopy

The FTIR test was conducted according to ASTM E 1252 [73] to evaluate the aging level of asphalt binder by tracking oxygen-containing molecules. The level of aging can be quantified based on the identification of oxygen-containing molecules within the structure of asphalt binder [66]. The oxygen containing group, carbonyl (C=O, a carbon atom double-bonded to an oxygen atom), can be traced in the wavenumber of 1,700 cm^{-1}. The carbonyl index (CI) was defined as a ratio of the bands' area around 1,700 cm^{-1} over total areas of spectra between 1,320 and 1,490 cm^{-1}, as expressed in Equation 9. Lower CI values are desired, as they indicate less aging effect on the asphalt binder and better cracking resistance. Figure 8 shows an example for FTIR spectra and CI calculation methodology. The FTIR spectrum for the tested asphalt binders were obtained using a Bruker Alpha FTIR spectrometer (Alpha), which uses a diamond single reflection attenuated total reflectance (ATR). An OPUS 7.2 data collection program was used for data analysis. The following settings were used for data collection: 16 scans per sample, spectral resolution 4 cm^{-1}, and wave number range 4000-500 cm^{-1}.

$$CI = \frac{Area\ of\ carbonyl\ band\ centered\ around\ 1700cm^{-1}}{\sum Area\ of\ spectral\ bands\ between\ 1320\ and\ 1490cm^{-1}} \quad (9)$$

Figure 8. FTIR spectrum for PG 76-22M asphalt binder at PAV



OGFC Mixture Design

OGFC mixture design was performed according to ASTM D 7064, "Standard Practice for Open-Graded Friction Course (OGFC) Mix Design," and Section 501, "Thin Asphalt Concrete Applications" of the 2016 "Louisiana Standard Specifications for Roads and Bridges." It specifies the use of conventional SBS-modified PG 76-22M asphalt binder with a minimum asphalt binder of 6.5%. Therefore, the PG 76-22M at 6.5% content was selected as the basis for the selection of optimum aggregate gradations in this study. Three candidate gradations (Gradation 1, Gradation 2, and Gradation 3) were selected based on Louisiana practices and literature [51, 74]; see Figure 9. Gradations 1 and 2 were selected from OGFC job mix formulas (JMFs) used in Louisiana, while Gradation 3 was selected from an Alabama study. The voids in coarse aggregate (VCA) parameter were calculated for each gradation to ensure a coarse aggregate skeleton with stone-on-stone contact according to ASTM D 7064. Dry-rodded unit weight for the coarse aggregate for each gradation (VCA_{drc}, Equation 10) was calculated in accordance with AASHTO T 19, "Standard Method of Test for Bulk Density ("Unit Weight") and Voids in Aggregate" [75] and AASHTO T 85, "Standard Method of Test for Specific Gravity and Absorption of Coarse Aggregate" [76]. OGFC mixtures (asphalt binder PG 76-22M at 6.5% content and aggregates with various gradations) were mixed and compacted using the Superpave gyratory compactor (SGC) at 50 gyrations. Loose mixture samples were used to determine the theoretical maximum density (G_{mm}) based on ASTM D 2041, "Standard Test Method for Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures" [77]. Using the bulk specific gravity and the theoretical maximum density, the percent air voids (V_a) and VCA of the compacted mixture (VCA_{mix}, Equation 10) were calculated. Gradation 1 was selected, as it produced an OGFC mixture that has a minimum of 18% air voids and $VCA_{mix} \leq VCA_{drc}$; see Figure 10. Note that OGFC mixtures with 7% asphalt binder content were also evaluated to ascertain the effects of asphalt binder contents on the durability of OGFC mixtures. Table 2 shows the OGFC mixture preparation parameters followed in this study.

$$VCA_{drc} = \frac{G_{CA}*\gamma_w - \gamma_s}{G_{CA}*\gamma_w} \tag{10}$$

$$VCA_{mix} = 100 - \left[\frac{G_{mb}}{G_{CA}} * P_{CA}\right] \tag{11}$$

where,

γ_w= unit weight of water;

γ_s= bulk density of the coarse aggregate fraction in the dry-rodded condition;

G_{CA} = bulk specific gravity of the coarse aggregate;
Gmb = bulk specific gravity of compacted mixture; and
P_{CA}= percentage of the coarse aggregate in the total mixture.

Figure 9. Three OGFC gradations selected for determining the optimum gradation

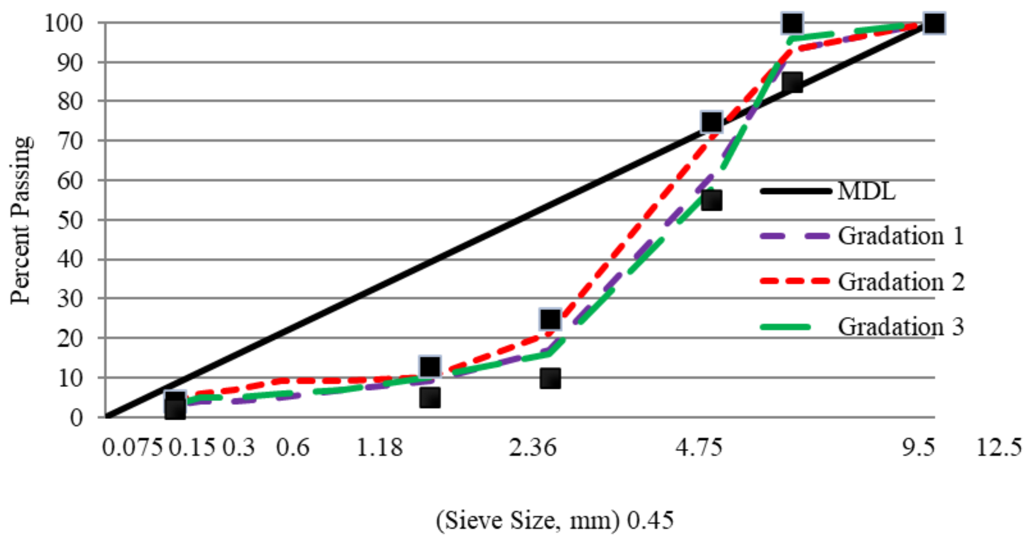


Figure 10. Volumetric parameters for three gradations

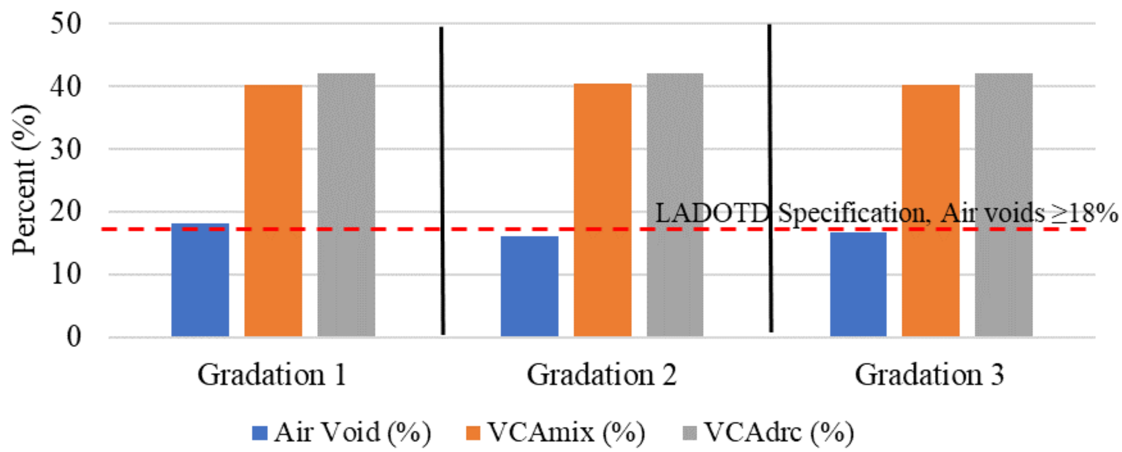


Table 2. Asphalt mixtures preparation parameters

Mixture ID	Modifier	Asphalt Binder Grade	Mixing Temperature, °C	Compaction Temperature, °C	STA, hrs.
76-22M	SBS	PG 76-22	165	155	2
76-22G	SBS/CRM	PG 76-22	165	155	2
88-28	SBS	PG 88-28	171	155	2
25%EA	EA	PG 70-22	121	121	1*
50%EA	EA	PG 70-22	121	121	1*

STA: Short term aging; PG: Performance grading; EA: Epoxy asphalt binder; SBS: Styrene-butadiene-styrene; CRM: Crumb Rubber; *: Diluted EA binders were aged for 1 hour based on manufacturer's instructions.

Asphalt Mixture Laboratory Tests

Table 3 presents the physical and mechanical tests conducted in the study. A brief description of each test is provided below.

Table 3. List of mechanical tests conducted on asphalt mixtures

Test designation	Testing temperatures (°C)	No. of replicates/ Sample Dimension: Dia. (mm) x Height (mm)	Engineering Properties	Protocols/Standards
Draindown	10 + mixing Temperature	3/ loose mixture	Draindown, %	ASTM D 6390
Permeability	25	3/ D150xH80	Coefficient of Permeability	NCAT
LWT	50	4/ D150 x H60	Rutting resistance	AASHTO T 324 (on unconditioned samples)
F/T+LWT	50	4/ D150 x H60	Moisture damage	AASHTO T 283 AASHTO T 324 (on F/T conditioned samples)
Modified Lottman (TSR)	25	6/ D150xH95	Moisture damage	AASHTO T 283 (on dry and F/T conditioned samples)
Cantabro abrasion loss	25	3/ D150 x H115	Durability	Tex-245-F

Note LWT: Load wheel track test; F/T: Freeze thaw conditioning; TSR: Tensile strength ratio D: Specimen diameter; H: Specimen height.

Draindown Test

This test was conducted in accordance with ASTM D 6390, "Standard Test Method for Determination of Draindown Characteristics in Uncompacted Asphalt Mixtures" [78]. This test procedure determines the amount of draindown in an uncompacted asphalt mixture (i.e., loose mixture) when the sample is held at elevated temperatures comparable to those encountered during the production, storage, transport, and placement of the mixture; see Figure 11. The test is particularly applicable to mixtures such as open-graded courses and stone matrix asphalt. A sample of asphalt mixture is placed in a wire basket, which is positioned on a plate or other suitable container of known weight. The sample, basket, and plate or container are placed in an oven for a specified amount of time at the production temperature. At the end of the heating period, the basket containing the sample is removed from the oven along with the plate or container, and the weight of the plate or container is determined. The amount of draindown is considered to be the portion of the material that separates itself from the sample as a whole and is deposited outside the wire basket. The material that drains may be composed of either a binder or a combination of binder and fine aggregate.

Figure 11. Draindown test

Permeability Test

This test was conducted in accordance with Florida Department of Transportation specification FM 5-565, "Measurement of Water Permeability of Compacted Asphalt Paving Mixtures" [79]. This test method covers the laboratory determination of the water conductivity of compacted asphalt mixture specimens with a diameter of 150 mm and a thickness of 95 mm. The measurement provides an indication of the water permeability of the asphalt mixture specimen. Figure 12 shows the falling head permeability test apparatus used to determine the rate of water flow through the specimen. Water in a graduated cylinder is allowed to flow through a saturated asphalt sample, and the interval of time taken to reach a known change in head is recorded. The coefficient of permeability of the asphalt sample is then determined.

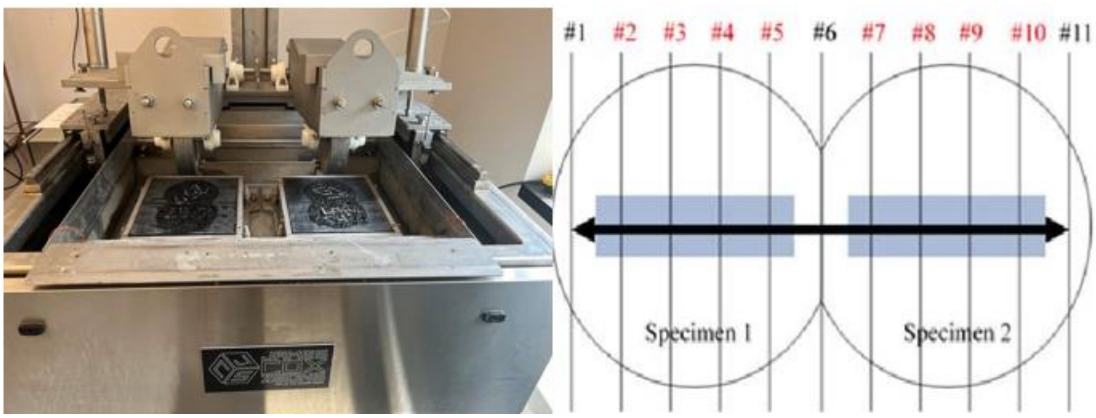
Figure 12. Permeability test

Loaded Wheel Track Test

This test was conducted in accordance with AASHTO T 324, "Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA)" [80]. This test is considered a torture test that produces damage by rolling a 703 N (158 lb.) steel wheel across the surface of cylindrical specimens (150 mm diameter by 60 mm thick) that are submerged in 50°C water for 20,000 passes at 52 passes per min. Four specimens (two for each wheel) are tested. Rut depth measurements are recorded during the duration of the test, and the rut depth at 20,000 cycles is recorded and used in the analysis. Rut depth measurements were collected at 11 locations across the cylindrical specimen. Then, rut depth measurements at four middle locations (2–5 and 7–10) for each location (left and right wheels) were averaged; see Figure 13. Additionally, the stripping inflection point (SIP) is calculated and reported as a measure of moisture damage for the mixtures evaluated.

Asphalt mixtures subjected to different levels of moisture conditioning (unconditioned and freeze-thaw cycles) were evaluated using the LWT. The freeze-thaw procedure was conducted according to AASHTO T 283 and adjusted according to ASTM D 7064, “Standard Practice for Open-Graded Friction Course (OGFC) Mix Design.” The samples were placed in a vacuum container filled with potable water at 25°C so that the samples had at least 25 mm of water above their surface. A vacuum of 10–26 in. Hg partial pressure (13–67 kPa absolute pressure) was applied for 5–10 min. The samples were then left submerged in water for 5-10 min., then transferred to a plastic bucket filled with potable water. It is noted that the samples should be under water at all times so that the specimen air voids are always filled with water. The bucket is then put in a freezer at (-18 ± 3°C) for at least 16 hrs. The samples are then placed in a water bath at 60°C for 24 hrs. and finally placed in a water bath at 25°C for 2 hrs. before testing.

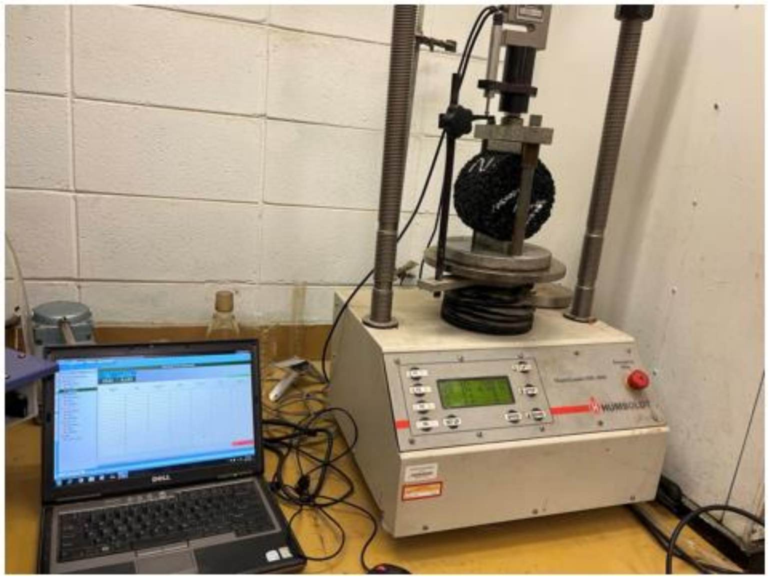
Figure 13. Loaded wheel track test



Modified Lottman Test

The modified Lottman test is conducted in accordance with AASHTO T 283, “Standard Method of Test for Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage” [81]. The test utilizes the freeze-thaw moisture conditioning cycle and subsequent indirect tensile strength (ITS) test to determine the effect of moisture conditions on the mixture’s indirect tensile strength. The procedure uses two sets of specimens compacted to 150 mm in diameter and 95 mm in thickness. The two sets consist of (1) a control set without conditioning and (2) a conditioned set with partial vacuum saturation and an optional freeze-thaw cycle. A split tensile test at 25°C was performed on each sample, and the indirect tensile strength of the conditioned samples was compared to the control group to determine the tensile strength ratio (TSR), which measures the effect of moisture on the indirect tensile strength; see Figure 14. A minimum TSR of 70% is often used as a standard criterion for OGFC mixtures [1].

Figure 14. Modified Lottman test



Cantabro Abrasion Loss Test

This test was conducted according to Tex-245-F, "Test Procedure for Cantabro Test" [82]. The test is conducted to evaluate the resistance to raveling in both unaged and aged specimens with dimensions of 150 mm in diameter and 115 mm in thickness. In the test, compacted specimens are put inside a Los Angeles Abrasion machine drum without steel balls, and the drum is turned for 300 revolutions for 10 min.; see Figure 15. The percentage of mass loss during this process is used to evaluate the resistance of asphalt mixtures to raveling; see Equation 12. For aging conditions, the compacted samples were aged in the oven for 5 days at 85°C (AASHTO R 30) to simulate the long-term aging of asphalt mixtures in the field.

$$Abrasion\ loss\ \% = \frac{A-B}{A} * 100 \qquad (12)$$

where,

Abrasion loss %= Cantabro abrasion loss, %;

A= initial weight of sample; and

B= final weight of sample.

Figure 15. Cantabro abrasion loss test

Statistical Analysis

A statistical analysis of variance (ANOVA) using Fisher's least significant difference at a 95% confidence level was performed on laboratory test data using the Statistical Package for Social Sciences (SPSS version 23) software. Based on the statistical analysis, the mixtures were characterized into statistical groupings that are represented by the letters A, B, C, and so forth and included in the figures, indicating statistically distinct performance from best to worst. Letter A denotes the best performance, followed by other letters ranked in order. A double letter designation (A/B) indicates that the difference in mean between the groups is insignificant. Further, error bars represent 95% confidence intervals from the mean and were included in the figures.

Discussion of Results

Asphalt Binder Test Results

Table 4 presents the performance grading test results for all of the asphalt binders evaluated. The rotational viscosity (RV) results show that all asphalt binders meet the requirements and have viscosities less than or equal to 3.0 Pa.s, except for PG 88-28. It is important to note that the PG 88-28 asphalt binder has a high SBS content and requires higher mixing temperatures, as shown in Table 2. The RV results indicate that the asphalt binders have adequate workability in the field. The performance grading (PG) was conducted according to AAHSTO M 320, “Standard Specification for Performance-Graded Asphalt Binder” [59], and the results show that all asphalt binders meet the requirements. It is worth noting that adding the EA binder improved the high PG grading of PG 67-22 base binder to PG 70-22.

Table 4. Asphalt binder performance grading results

Test		Spec [59]	PG67-22	PG76-22M	PG76-22G	PG88-28	25%EA	50%EA
Tests on Original Binder								
Rotational Viscosity, 135°C, Pa.s		3.0^-	0.8	2.8	2.3	3.7	1.1	1.2
DSR, G*/Sin(δ), kPa	67°C	1.00^+	1.50	-	-	-	1.98	2.16
	70°C	1.00^+	0.92	4.50	3.51	4.82	1.26	1.39
	76°C	1.00^+	-	2.58	2.01	3.56	-	-
	82°C	1.00^+	-	-	-	2.76	-	-
	88°C	1.00^+	-	-	-	2.17	-	-
Tests on RTFO								
DSR, G*/Sin(δ), kPa	67°C	2.20^+	3.25	-	-	-	4.69	5.92
	70°C	2.20^+	1.92	-	-	-	3.02	4.98
	76°C	2.20^+	-	2.95	3.14	5.03	1.51	1.90
	82°C	2.20^+	-	1.76	1.81	3.79	-	-
	88°C	2.20^+	-	-	-	2.94	-	-
	94°C	2.20^+	-	-	-	1.86	-	-
MSCR, 67°C, J_{nr}, 3.2kPa^{-1}, 0.5^- for PG 76-22M		0.5^-	3.3	0.3	0.4	0.02	1.9	2.0
MSCR, %Recovery, 3.2kPa^{-1}, 67°C		-	0.5	65.4	56.8	96.5	2.0	1.5
Tests on (RTFO + PAV)								
DSR, 26.5°C, G*Sin(δ), kPa, 5000- for PG 67-22; 6000^- for PG 76-22M		5000^- 6000^-	3290	3060	2905	721	3780	4020
BBR, Creep Stiffness, MPa, -12°C		300^-	165	181	167	103	203	212
BBR, m-value, -12°C		0.300^+	0.320	0.344	0.353	0.339	0.342	0.352
BBR, Creep Stiffness, MPa, -18°C		300^-	333	380	381	295	430	461

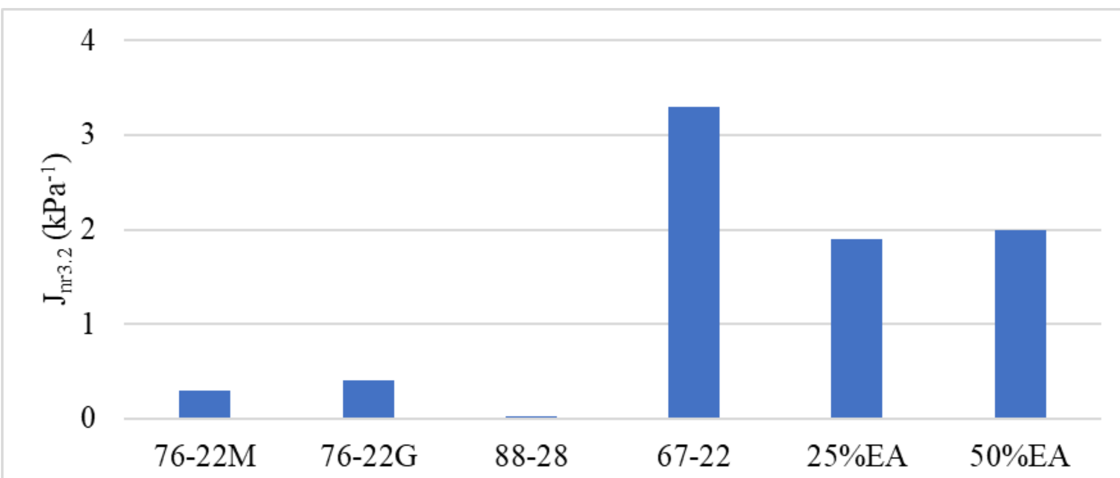
Test	Spec [59]	PG67-22	PG76-22M	PG76-22G	PG88-28	25%EA	50%EA
BBR, m-value, -18°C	0.300+	0.259	0.255	0.271	0.293	0.274	0.262
Actual PG Grading		PG 67-22	PG 76-22	PG 76-22	PG 88-28	PG 70-22	PG 70-22

Note: DSR: Dynamic shear rheometer; MSCR: Multiple stress creep recovery; J_{nr}: Non-recoverable creep compliance; BBR: Bending beam rheometer; PG: Performance grading; RTFO: Rolling thin film oven; PAV: Pressure aging vessel.

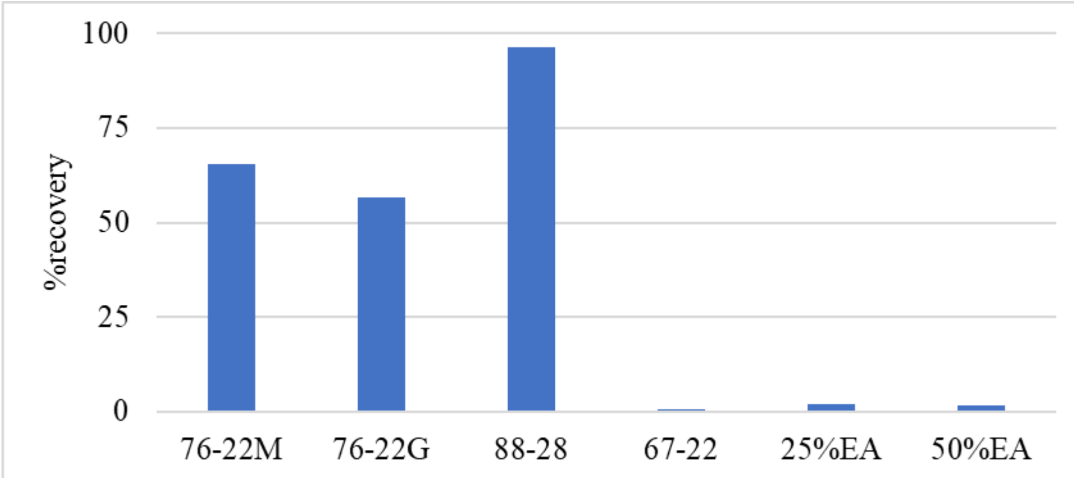
Multiple Stress Creep Recovery (MSCR) Test Results

Figure 16 presents the MSCR results at the 3.2 kPa stress level. Generally, MSCR tests are conducted at two stress levels (0.1 kPa and 3.2 kPa), but only the 3.2 kPa stress level was selected, as it was more critical and better reflected the rutting potential of asphalt binders. The primary factors selected from this test to compare the high-temperature rheological properties of asphalt binders are non-recoverable creep compliance (J_{nr}) and percent recovery (%recovery). The combination of lower J_{nr} and higher %recovery is desired, as it indicates higher rutting resistance as well as better recovery under repeated shear loads. The PG 67-22 base binder showed the highest J_{nr} and lowest %recovery values, which indicates the lowest resistance to rutting among the asphalt binders evaluated. Further, the EA binder improved the base asphalt binder PG 67-22 results by decreasing its J_{nr} as well as increasing its %recovery. Finally, PG 76-22G, PG 76-22M, and PG 88-28 had the lowest J_{nr} and highest %recovery, indicating their superior rutting resistance. It is worth noting that MSCR tests were conducted on RTFO-aged asphalt binders, which might not be adequate for capturing the high-temperature rheological properties of the diluted EA binder because of the incomplete chemical reaction between the EA and base binder.

Figure 16. Multiple stress creep recovery results at 3.2kPa
(a) nonrecoverable creep compliance (J_{nr}) and (b) %recovery



(a)

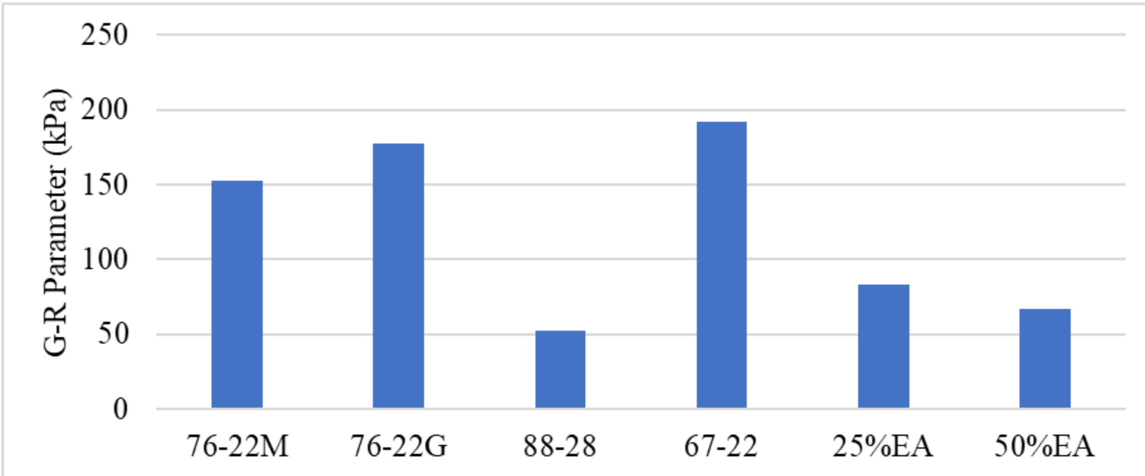


(b)

Frequency Sweep (FS) Test Results

Figure 17 shows the frequency sweep test results as measured by the Glover-Rowe (G-R) parameter. A lower G-R value is desired, as it indicates better cracking resistance at intermediate temperatures [68]. PG 88-28 had the lowest G-R value, followed by 50%EA, 25%EA, PG 76-22M, PG 76-22G, and PG 67-22 asphalt binders. EA-diluted binders showed a significant improvement in G-R values compared to the PG 67-22 base binder. The EA-diluted binder shows its excellent rheological properties when fully cured. Thus, tests conducted on PAV-conditioned samples are expected to show better results compared to tests conducted on unaged or RTFO-aged samples.

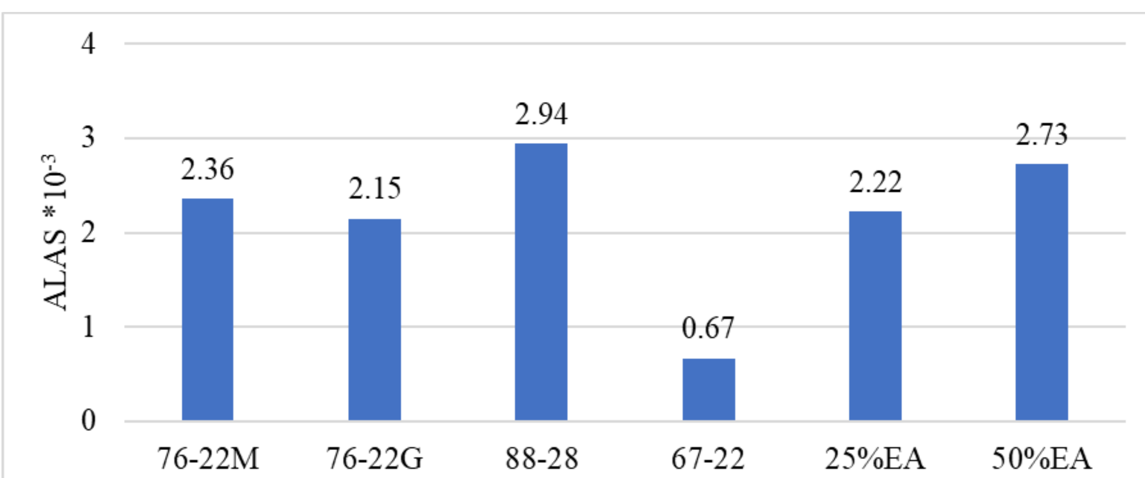
Figure 17. Frequency sweep results, G-R parameter



Linear Amplitude Sweep (LAS) Test Results

Figure 18 shows the linear amplitude sweep (LAS) test results, as measured by the A_{LAS} parameter. A higher A_{LAS} value is desired, as it indicates better cracking resistance [66]. Similar to the results shown in the frequency sweep test, the linear amplitude sweep test results show the same trend. PG 88-28 exhibited the highest A_{LAS} value, followed by 50%EA, 25%EA, PG 76-22M, PG 76-22G, and PG 67-22 asphalt binders. EA-diluted binders showed a significant improvement in A_{LAS} values compared to the PG 67-22 base binder.

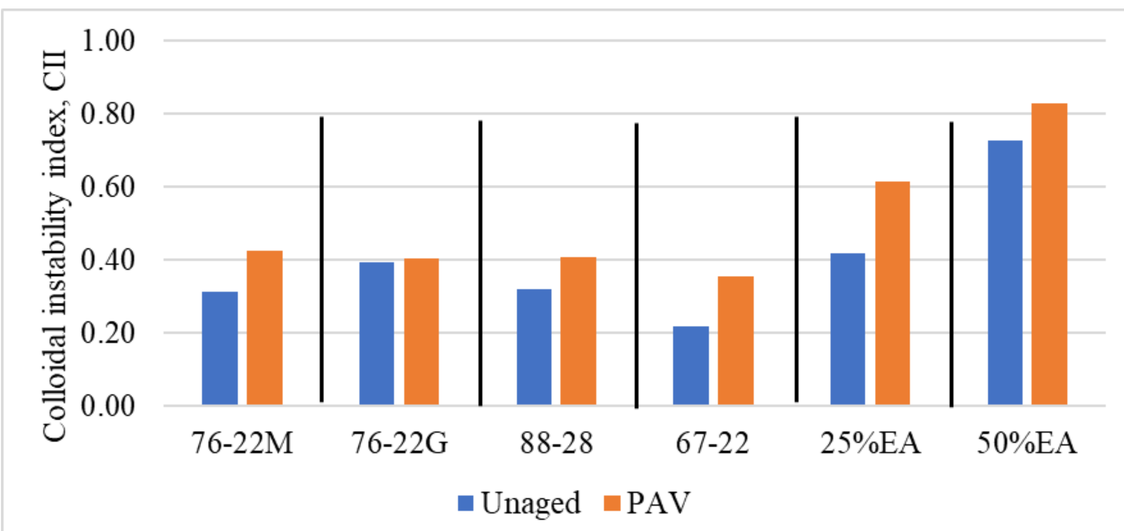
Figure 18. Linear amplitude sweep results, A_{LAS} parameter



SARA Fractions Results

Figure 19 shows the colloidal instability index (CII) values for all asphalt binders at both unaged and PAV aging levels. The CII is a measure of the tendency of asphalt binder to undergo colloidal (i.e., particle) instability or separation. Colloidal instability in asphalt binders can result from the agglomeration of asphaltenes and other components, especially after aging, leading to the formation of larger particles and potentially causing issues such as phase separation and poor asphalt binder performance [83, 84, 85, 86, 87, 88]. Therefore, a lower CII value is desired, as it suggests better resistance to cracking. As seen in Figure 19, the CII value increases with aging for all binder types. The PG 67-22 base binder showed the least CII value, followed by PG 76-22M, PG 88-28, PG 76-22G, 25%EA, and 50%EA. Nonetheless, it might be more meaningful to compare the percent increase in CII instead. For example, 25%EA and 50%EA binders showed a lower percent increase in CII values compared to base binder PG 67-22 after PAV. This shows the excellent aging resistance properties of EA binder and suggests better durability. Similarly, PG 88-28, PG 76-22M, and PG 76-22G exhibited a lower percent increase in CII values as compared to base binder PG 67-22.

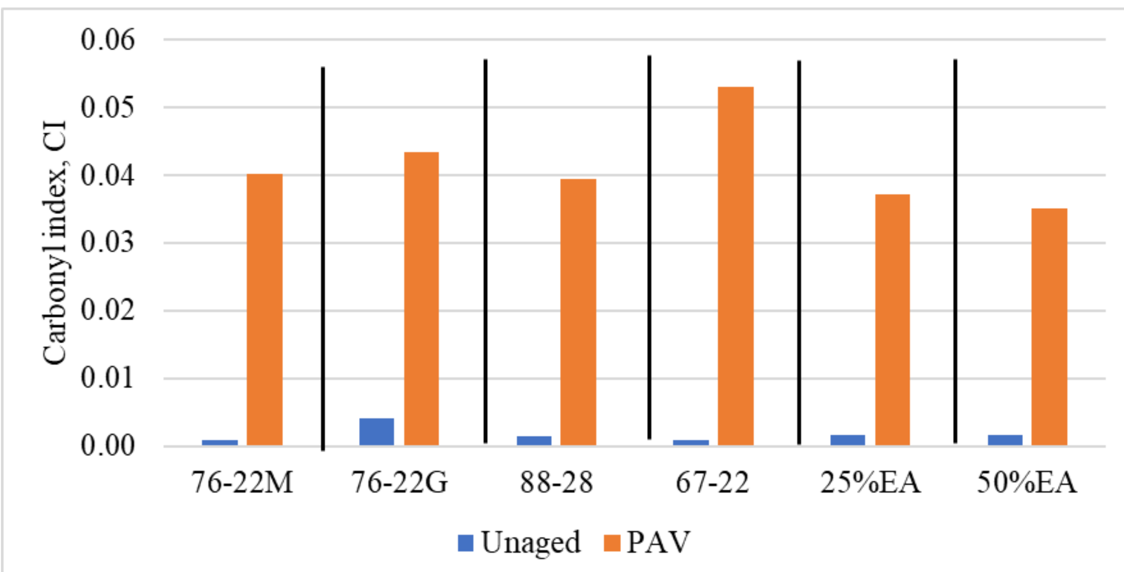
Figure 19. Colloidal instability index results



Fourier Transform Infrared (FTIR) Spectroscopy Test Results

Figure 20 presents the carbonyl index (CI) results for all asphalt binders at both unaged and PAV aging levels. The carbonyl index is a way to quantify the level of oxidation in the binder. A higher carbonyl index indicates a higher degree of oxidative aging, which can adversely impact the binder's durability. Therefore, a lower CI value is desired, as it suggests less oxidation because of aging and more crack-resistant and durable asphalt binders [89, 90, 91, 92, 93]. The CI values for the unaged asphalt binders were approximately zero because of the absence of oxidation expected for the unaged samples. However, a significant increase in CI values was observed after PAV aging. This observation can be attributed to the sulfoxide and carbonyl functional groups developed after aging. 25%EA and 50%EA binders significantly lowered CI values compared to the PG 67-22 base binder and exhibited the least CI values among all asphalt binders tested. The aforementioned observation highlights the excellent aging resistance properties of EA binders. Similarly, PG 88-28, PG 76-22M, and PG 76-22G exhibited a lower percent increase in CI values compared to the PG 67-22 base binder.

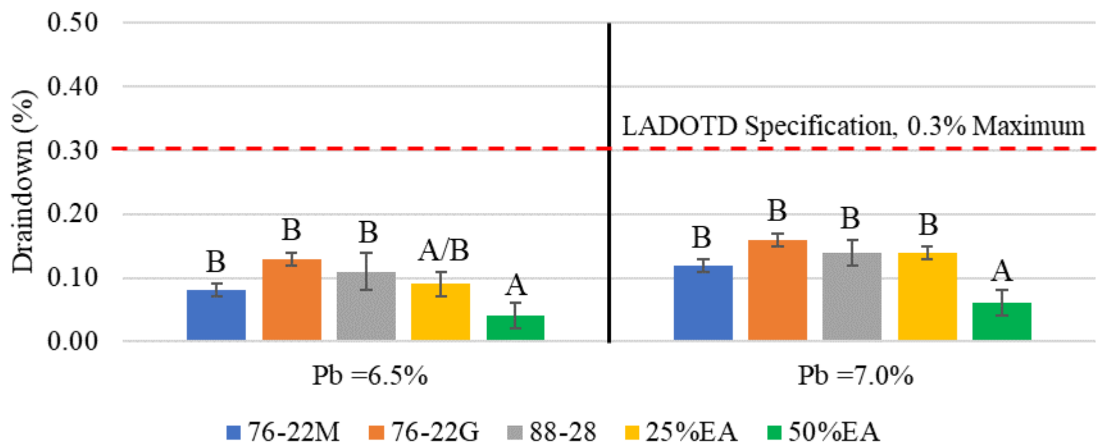
Figure 20. Carbonyl index results



Asphalt Mixture Test Results

Figure 21 presents the draindown test results for two asphalt binder contents, 6.5% and 7.0%. The coefficient of variation (CoV) for draindown test results ranged between 2% and 8%, with an average of 5%. The draindown test temperatures for mixtures containing PG 76-22M, PG 76-22G, PG 88-28, and diluted-EA binders were 178ºC, 175ºC, 181ºC, and 131ºC, respectively. Results showed that all mixtures exhibited draindown values less than the maximum allowable amount of 0.3%, implying that the OGFC mixtures have acceptable draindown limits during mixture design and/or during field production [51]. Further, mixtures with an asphalt content of 7.0% tended to have slightly higher draindown values than mixtures with an asphalt content of 6.5%. Moreover, mixtures with 50%EA binder had significantly lower draindown values at the two asphalt contents compared to other mixtures.

Figure 21. Draindown test results

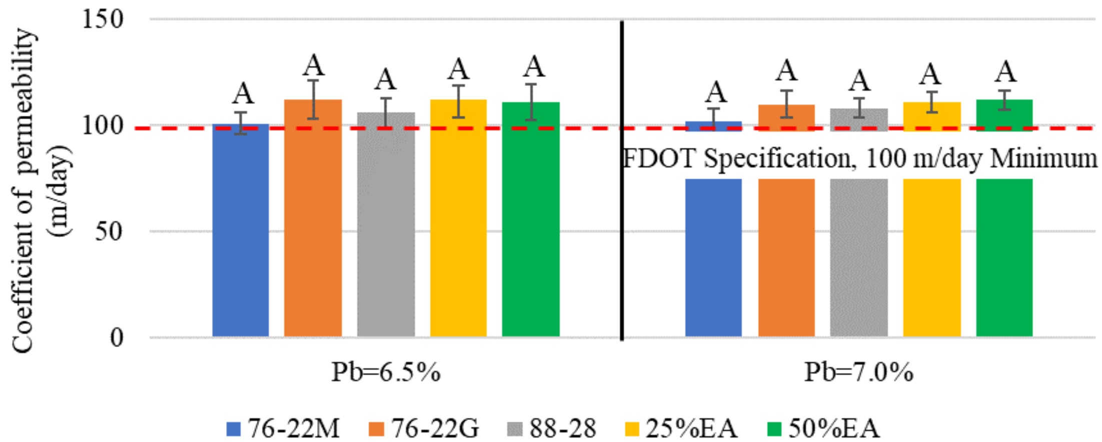


Permeability Test Results

Figure 22 presents the permeability test results for the mixtures evaluated at two asphalt binder contents, 6.5% and 7.0%. The CoV of the coefficient of permeability test results ranged between 7% and 16%, with an average of 12%. The Florida DOT specifies a minimum coefficient of permeability value of 100 m/day for OGFC mixtures [78]. All mixtures evaluated met the minimum requirement of 100 m/day and showed statistically similar results. This observation can be attributed to the fact that permeability is primarily governed by aggregate structure (such as gradation, NMAS, etc.) and not asphalt binder type

[94, 95, 96, 97, 98]. It is noted that the mixtures evaluated had similar aggregate structures. Thus, permeability results were expected to be similar; see Figure 22.

Figure 22. Permeability test results

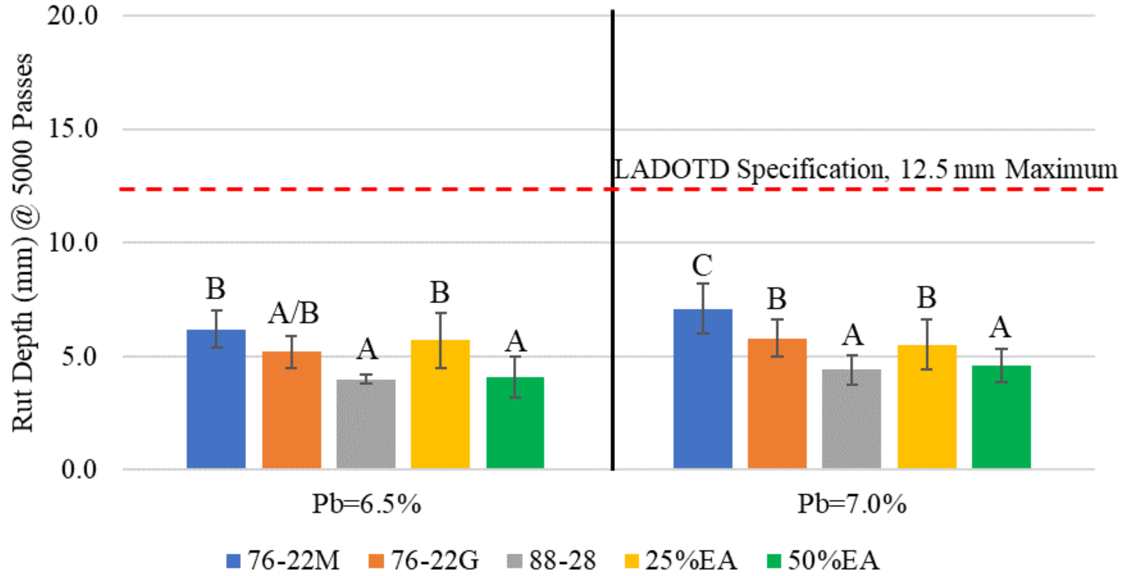


Loaded Wheel Track (LWT) Test Results: Permanent Deformation

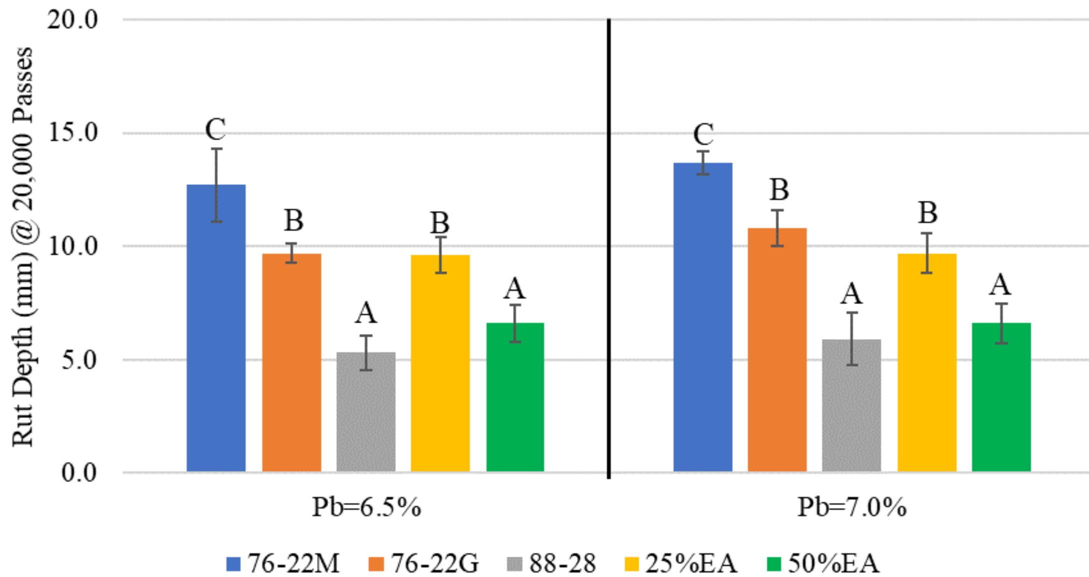
Figure 23 shows LWT rut depth values at 5,000 and 20,000 passes for unconditioned OGFC mixtures. The CoV of LWT results ranged between 8-17%, with an average of 14%. Louisiana DOTD specifies a maximum rut depth of 12.5 mm at 5,000 passes [50]. All mixtures evaluated met this maximum rut depth requirement; see Figure 23a. However, OGFC mixtures containing PG 76-22M showed significantly higher rut depth at 5,000 passes compared to other mixtures evaluated at the two asphalt contents. It is worth noting that all mixtures exhibited statistically similar rut depth at 5,000 passes, except the one containing PG 76-22M; see Figure 23a. OGFC mixtures containing PG 76-22M showed significantly higher rut depth at 5,000 passes compared to OGFC mixtures with PG 88-28 and 50%EA at $P_b = 6.5\%$ and significantly higher rut depth at 5,000 passes compared to all mixtures at $P_b = 7.0\%$; see Figure 23a.

Figure 23b presents LWT rut depth values at 20,000 passes. At this level of high temperature damage, the effect of asphalt binder type was pronounced. Specifically, mixtures containing PG 88-28 and 50%EA binders showed the lowest rut depth values, followed by those with 25% EA, PG 76-22G, and PG 76-22M at both asphalt contents. Increased levels of SBS and EA modifications were the primary contributors to this positive high-temperature performance [1, 17].

Figure 23. LWT test results for unconditioned samples: (a) 5,000 passes and (b) 20,000 passes



(a)



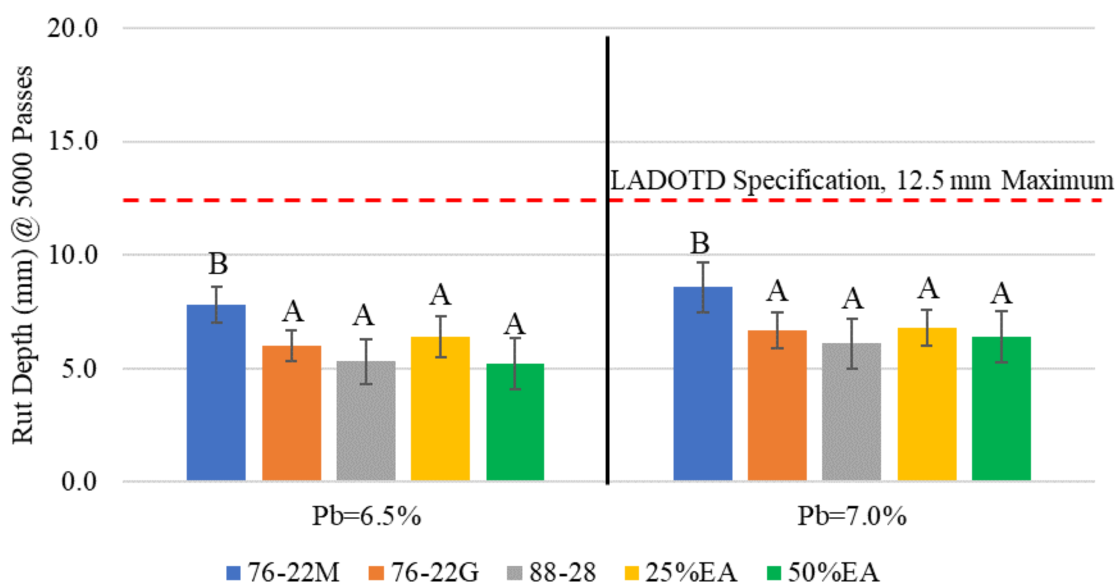
(b)

Loaded Wheel Track (LWT) Test Results: Moisture Damage

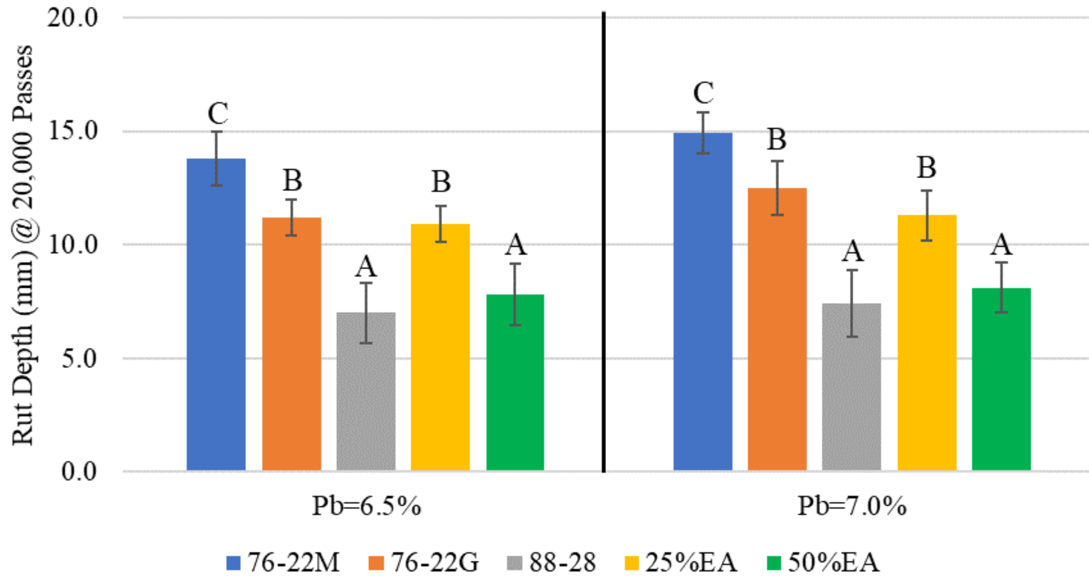
Figure 24 shows the LWT rut depth values at 5,000 and 20,000 passes for moisture conditioned samples (i.e., one freeze-thaw cycle). The CoV of the test results ranged between 7-17%, with an average of 12%. Similar to the LWT permanent deformation test, all mixtures evaluated exhibited rut depth values less than 12.5 mm at 5,000 passes and are considered to be moisture-resistant; see Figure 24a. Further, the trend of the effect of asphalt binder type was similar to that of LWT permanent deformation, in which PG 76-22M showed significantly higher rut depth values at 5,000 passes compared to other mixtures evaluated at the two asphalt contents. However, mixtures with the remaining asphalt binder types had statistically similar LWT rut depths at 5,000 passes; see Figure 24a.

Figure 24b shows LWT rut depth values at 20,000 passes for moisture conditioned samples. Similar to the LWT test for the unconditioned samples, the effect of the asphalt binder type was distinct. Specifically, mixtures containing PG 88-28 and 50%EA binders showed the lowest rut depth, followed by ones with 25%EA, PG 76-22G, and PG 76-22M at both asphalt contents. Increased levels of SBS and EA modifications were the primary contributors to this positive moisture damage performance [1, 17]. It is noted that all studied OGFC mixtures exhibited a stripping inflection point (SIP) of 20,000 passes for both unconditioned and F/T moisture-conditioned samples, indicating that all of the studied mixtures were moisture-resistant.

Figure 24. LWT test results for moisture conditioned samples (one freeze-thaw cycle): (a) 5,000 passes and (b) 20,000 passes



(a)



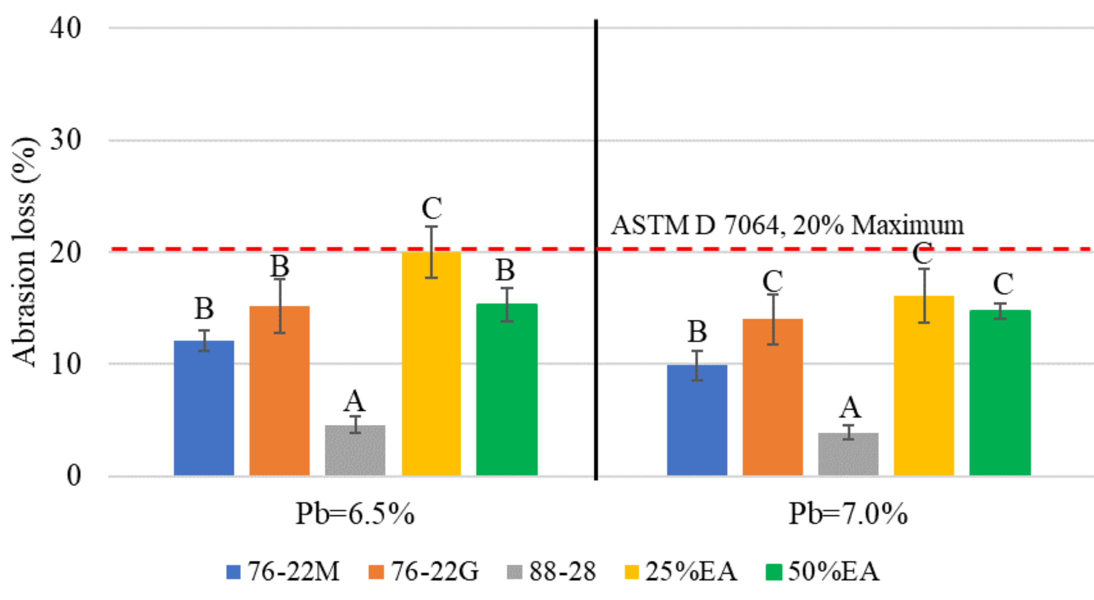
20.0
15.0
10.0
5.0
0.0
Rut Depth (mm) @ 20,000 Passes
C
B
A
B
A
C
B
A
B
A
Pb=6.5%
Pb=7.0%
76-22M
76-22G
88-28
25%EA
50%EA

(b)

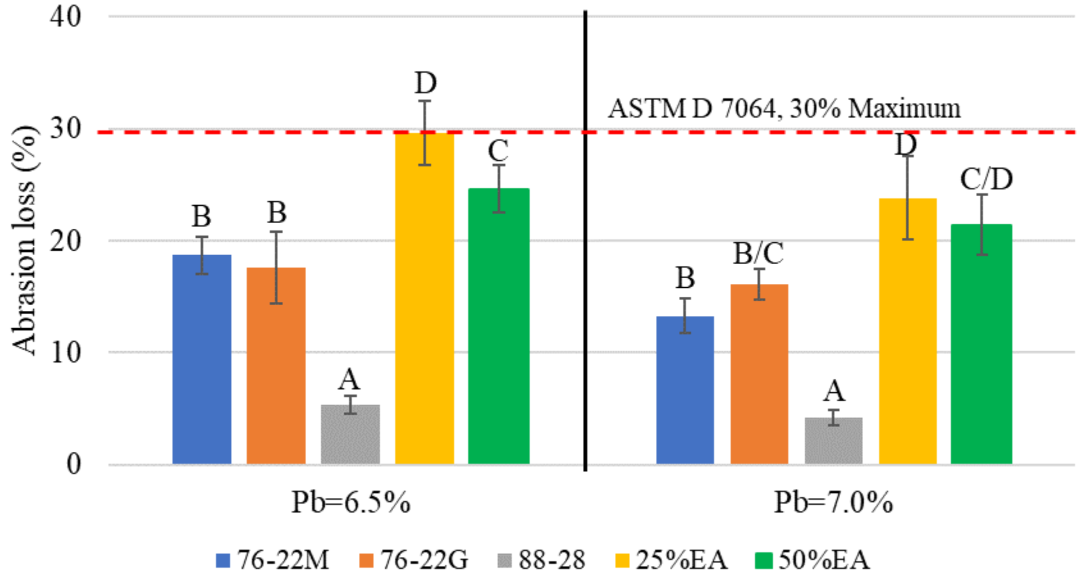
Cantabro Abrasion Loss Test Results

Figure 25 shows the Cantabro abrasion loss test results at two aging levels, unaged and aged (5 days, 85°C), and two asphalt contents (6.5% and 7.0%). The CoV of Cantabro abrasion loss test results ranged between 6-18%, with an average of 14%. Lower abrasion loss is desired for durable mixtures [4, 5]. Maximum abrasion loss for OGFC mixtures is specified at 20% and 30% for unaged and aged samples, respectively [49]. All mixtures evaluated met the requirements for both unaged and aged conditions; see Figure 25 [49]. As expected, aged (5 days at 85°C) mixtures with lower asphalt content (6.5%) exhibited higher abrasion loss. The mixture containing PG 88-28 was the most durable (i.e., significantly lower abrasion loss) compared to the other mixtures evaluated. At an asphalt content of 6.5% and in an unaged condition, mixtures containing PG 76-22M, PG 76-22G, and 50%EA binders showed statistically similar abrasion loss. However, at an asphalt content of 6.5% and aged condition, mixtures containing conventional PG 76-22M exhibited improved abrasion loss compared to similar mixtures with PG 76-22G, 25%EA, and 50%EA. For EA binders, as the dilution rate increased from 25 to 50%, the abrasion loss tended to generally decrease significantly; see Figure 25. Wu et al. reported a similar observation [26].

Figure 25. Cantabro abrasion loss test results: (a) unaged and (b) aged samples (5 days)



(a)



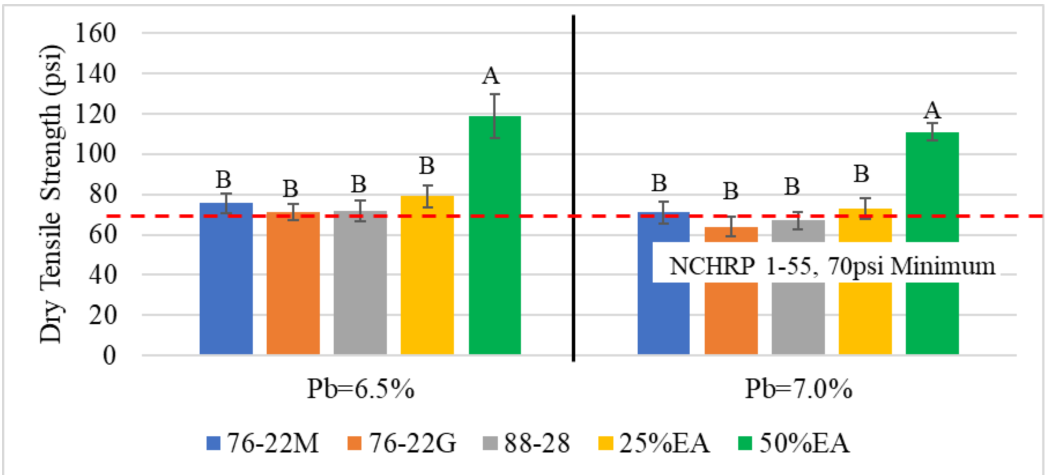
(b)

Modified Lottman Test Results

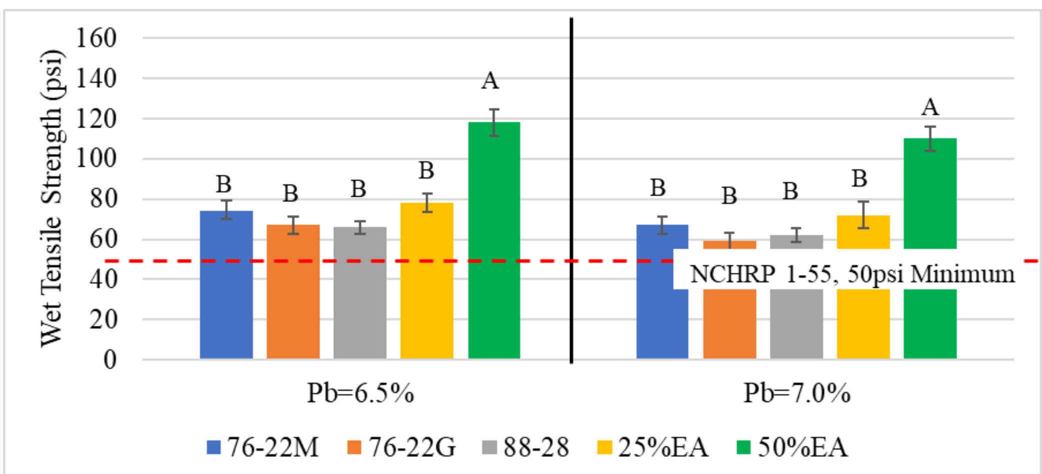
Figure 26 shows the Modified Lottman test results for two asphalt binder contents (6.5% and 7.0%). The CoV of modified Lottman test results varied between 4-13%, with an average of 8%. Higher dry and wet strength values as well as a higher tensile strength ratio (TSR) are desired for better performance and moisture damage resistance. Minimum tensile strengths for OGFC mixtures are recommended by NCHRP Report 1-55 at 70 and 50 *psi* for dry- and wet-conditioned specimens, respectively [3]. Further, the report recommends a minimum TSR of 70% for adequate moisture damage resistance. All mixtures evaluated met the recommended tensile strength values for both dry and wet conditions, except for the mixtures with PG 76-22G and PG 88-28 at P_b = 7.0% in the dry condition; see Figure 26. The 7% asphalt binder content resulted in slightly lower strength values, but the results were comparable. Additionally, all mixtures resulted in comparable strength results for dry and wet conditions at the two asphalt contents, except for the mixture with 50%EA, which had a significant strength increase under all testing conditions. This observation indicates that incorporating a higher dosage of EA binder (50%) is expected to significantly improve tensile strength and moisture damage resistance. Finally, the TSR values ranged from 91.9-99.6%, which shows that all mixtures exhibited much higher values compared to the 70% minimum value recommended by NCHRP Report 1-55 [3]. It is noted that the mixtures with

EA binder exhibited the highest TSR values, indicating the best moisture damage resistance among all the mixtures evaluated.

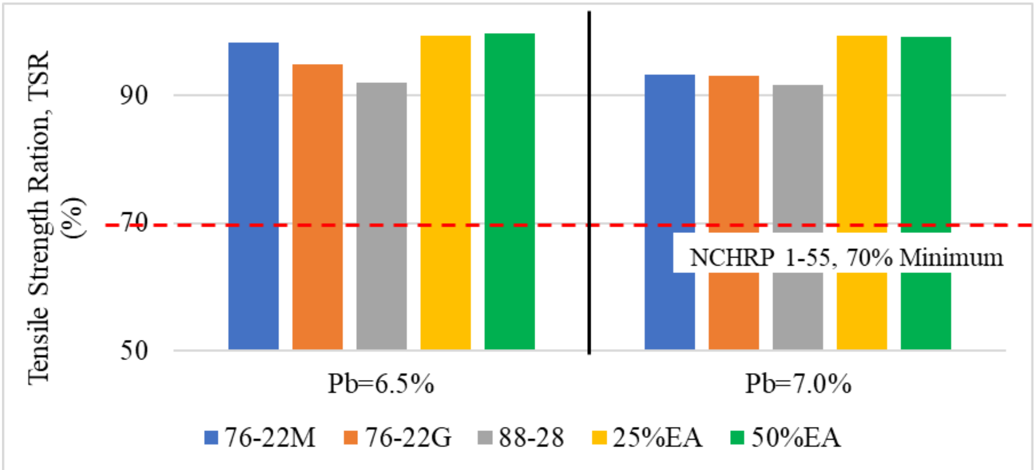
Figure 26. Modified Lottman results for (a) dry strength, (b) wet strength, and (c) TSR%



(a)



(b)



(c)

Cost-Effectiveness Ratio for Asphalt Mixtures

The cost-effectiveness ratio (CER), as shown in Equation 13, was computed for the higher asphalt content (P_b=7.0%) mixtures .These mixtures were aged at 85°C for 15 days to evaluate their long-term durability. To capture the long-term durability performance of EA mixtures, aging should extend beyond the recommended 5 days (ASTM D 7064) [49]. However, it is unknown how the extended aging condition correlates to field aging, which requires further investigation.

$$CER = \frac{Ci}{Ei} \tag{13}$$

$$Ei = \frac{30\% - CL\% \text{ at } 15 \text{ days}}{CL\% \text{ at } 15 \text{ days}} \tag{14}$$

Where,
CER= cost-effectiveness ratio of each asphalt mixture ($/ton);
Ci= cost per ton of each asphalt binder ($/ton);
Ei= percent change in Cantabro abrasion loss (%CL); and
30%= failure criterion of Cantabro abrasion loss (ASTM D 7064).

A lower CER value is preferred because it indicates that a particular technique for enhancing the durability of OGFC mixtures is cost-effective. The average cost per ton of PG 76-22M, PG 76-22G. PG 88-28, 25%EA, and 50%EA mixtures were $755/ton, $735/ton, $850/ton, $2030/ton, and $3475/ton, respectively, according to DOTD and EA manufacturers.

Figure 27 shows the effect of three aging durations (unaged, 5 days, 85°C, and 15 days, 85°C) on Cantabro abrasion loss test results at an asphalt content of 7.0%. It is noted that mixtures with an asphalt content of 7.0% showed the greatest improvement in Cantabro abrasion loss values and were subsequently subjected to additional aging conditions (15 days instead of the conventional 5 days) presented in Figure 27. In general, abrasion loss increased with an increase in aging duration for all mixtures evaluated, except for mixtures with EA binders. Further, the abrasion loss values for the mixtures used for the cost-effective analysis were below the recommended maximum of 30% after 15 days of aging at 85°C. However, two clusters of responses were observed; see Figure 27. Mixtures containing PG 88-28 had the lowest Cantabro abrasion loss compared to other mixtures evaluated. The high SBS polymer modification is the primary contributor to this improvement in mixtures' durability [17]. It is noted that mixtures containing EA binders showed improvement in abrasion loss as the aging duration increased from 5 to 15 days. This improvement may be attributed to the continued chemical reaction between the EA binder and the base asphalt binder at the aging temperature (85°C), as reported by others [26].

Figure 27. Cantabro abrasion loss test results for all aging conditions, P_b 7.0%

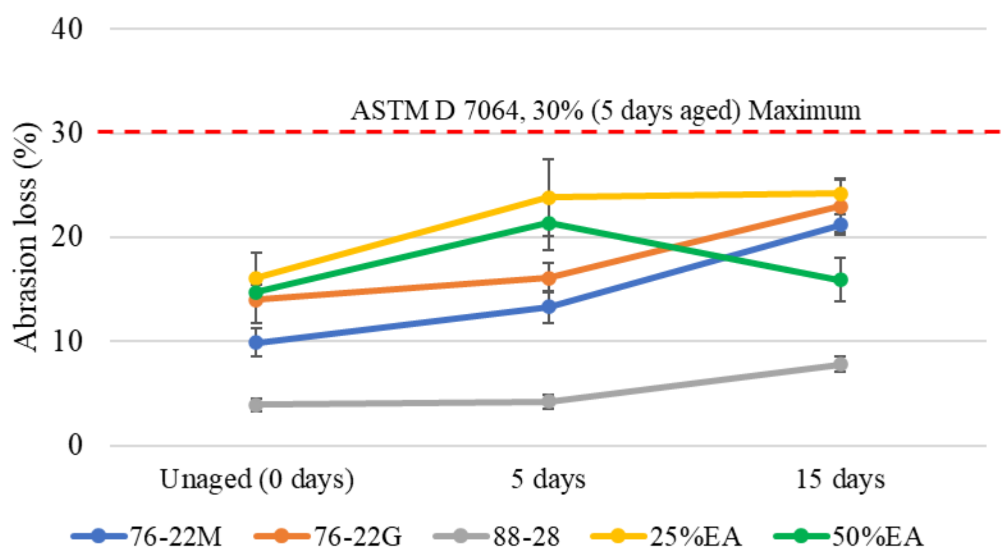


Figure 28 summarizes the results of the cost-effectiveness ratio for all mixtures aged for 15 days at P_b 7.0%. The mixture modified with PG 88-28 showed the lowest CER value (most cost-effective), followed by the OGFC mixtures prepared with PG 76-22M (control), PG 76-22G, 50%EA, and 25%EA. Further, CER results indicate that the 50%EA OGFC mixture was cost-effective compared to the 25%EA OGFC mixture, despite the increased cost associated with the higher EA binder.

Figure 28. Cost-effectiveness results (15 days aged)

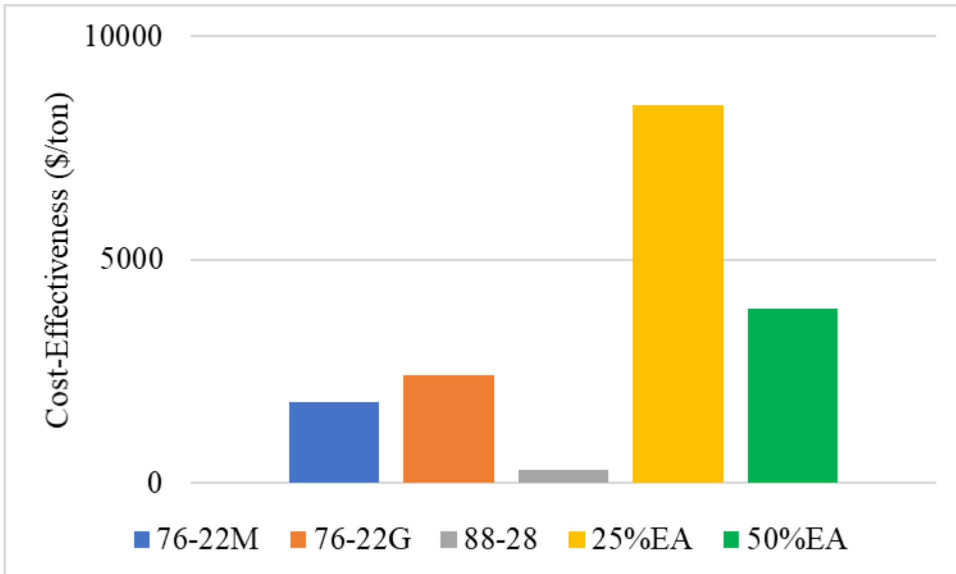
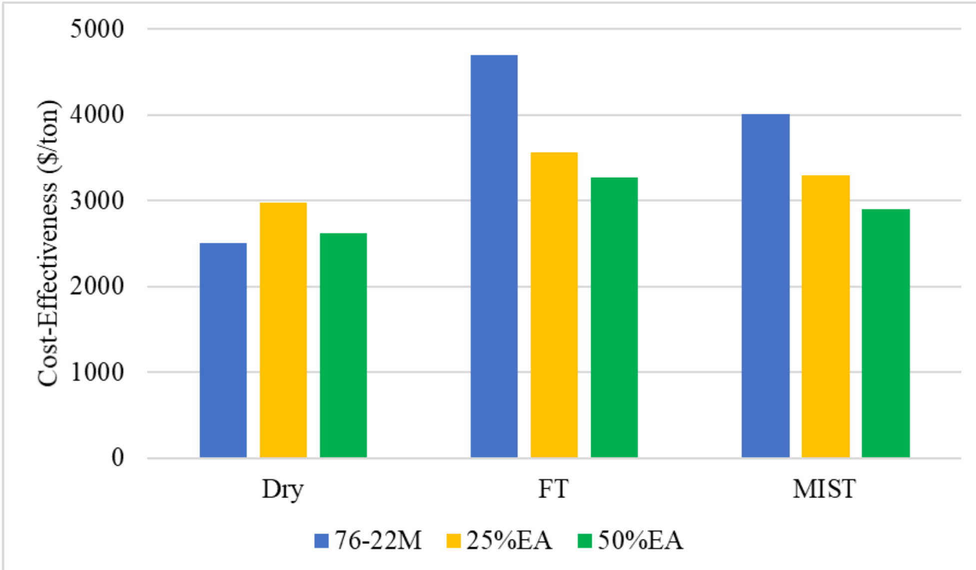


Figure 29 summarizes the results of the cost-effectiveness analysis for 76M and EA mixtures evaluated at 30 days of aging at dry, FT, and MIST conditioning for P_b=7.0%. Mixture with PG 76-22M (control) showed the highest effectiveness (lowest CER) at the dry condition followed by mixtures with EA. However, this trend has changed after moisture conditioning (FT and MiST). Mixtures with 50%EA and 25%EA binders exhibited improved effectiveness of 30.4% and 24.4% for FT conditioning, and 18.1% and 27.7% for MiST conditioning, respectively, compared to the control mixture. Therefore, CER results suggest that EA mixtures are expected to have higher effectiveness despite their higher initial costs compared to the conventional mixture used in Louisiana. Further, additional aging and moisture conditioning might be required to show the effect of asphalt binder type on the effectiveness of OGFC mixtures.

Figure 29. Cost-effectiveness results (30 days aged)



Conclusions

The objective of this study was to evaluate the durability and performance of OGFC mixtures containing various types of asphalt binder. The compatibility of the EA binder with multiple asphalt binder sources was first assessed, followed by rheological and chemical characterization for all asphalt binder types used. OGFC mixtures were laboratory produced with NMAS of 12.5 mm and five asphalt binder types, namely conventional SBS-modified PG 76-22M asphalt binder; high-SBS content PG 88-28 asphalt binder; diluted epoxy-modified asphalt (EA) binder prepared at two dosage rates (25% and 50% by weight of asphalt binder); and a hybrid PG 76-22G asphalt binder prepared with SBS and crumb rubber modifier (CRM). These mixtures were evaluated at two asphalt contents, namely 6.5% and 7.0%. The experimental plan consisted of four laboratory experiments. The first experiment was conducted to select the most compatible base asphalt binder to dilute the EA binder based on microscopic imaging and chemical compatibility testing. The second experiment was performed to characterize the rheological and chemical properties of the selected asphalt binder types. The third experiment was conducted to develop the mix design for OGFC mixtures. The final experiment was conducted to characterize the physical and mechanical performance of the OGFC mixtures. The following summarizes the research findings:

Asphalt Binder Characterization

- Chemical compatibility and microscopic analyses showed that base asphalt binder source 1 was most compatible when diluted with EA binder. Results showed that asphalt binders with similar performance grading (PG) but different sources may show different compatibilities. Therefore, it is recommended to evaluate the compatibility of the base asphalt binder used to dilute EA binder.
- The asphalt binder types evaluated had different performance grades based on their modifications. For example, the high-SBS content asphalt binder was graded as PG 88-28. Moreover, EA binder improved high-temperature grading of PG 67-22 asphalt binder for both dosages (25% and 50%).
- MSCR results showed that epoxy modification improved the creep compliance (decrease in J_{nr}) and slightly enhanced the elastic recovery (increase in %recovery). However, results were not as pronounced when using SBS and hybrid SBS/CRM modifiers.

- Results from FS and LAS tests showed that EA and PG 88-28 binders exhibited the lowest G-R and A_{LAS} values, indicating an improved crack resistance and long-term durability.
- Chemical characterization of asphalt binder types performed by SARA fractions and FTIR tests showed that EA binders had the lowest percent increase in CII and the lowest CI values, indicating better cracking resistance and the highest aging resistance, respectively.

Asphalt Mixture Characterization

- Minimum air voids and VCA parameter were key factors in determining a suitable mix design for OGFC mixtures. Based on these results, Gradation 1 had the optimum aggregate gradation and was selected.
- All OGFC mixtures evaluated exhibited draindown values less than the maximum specified value of 0.3%, implying that the OGFC mixtures had acceptable draindown limits during mixture design and/or during field production. The mixture with 50%EA binder had significantly lower draindown values compared to other mixtures.
- All OGFC mixtures evaluated met the minimum permeability requirement of 100 m/day and showed statistically similar results. This result indicates that permeability is primarily governed by the aggregate structure, which was similar for all mixtures.
- All OGFC mixtures evaluated complied with DOTD specification of maximum LWT rut depth requirement of 12.5 mm at 5,000 passes. Mixtures containing 50%EA and PG 88-28 binders showed the lowest rut depth at 20,000 passes, followed by those with 25% EA, PG 76-22G, and PG 76-22M.
- All OGFC mixtures evaluated were found to be moisture resistant, as they exhibited LWT rut depth values of less than 12.5 mm at 5,000 passes after freeze-thaw moisture conditioning. Also, results from the Modified Lottman test exhibited similar findings, as measured by their high TSR values.
- Studied OGFC mixtures complied with the ASTM D 7064 specification of maximum abrasion loss requirements of 20% and 30% for unaged and aged samples (5 days), respectively. Mixtures containing 25%EA and 50%EA binders showed an improvement in abrasion loss as aging duration increased from 5 to 15 days.

- Cost-effectiveness ratio (CER) results showed that EA mixtures have higher effectiveness compared to the conventional OGFC mixture with PG 76-22M when tested for 30 days aged moisture-conditioned Cantabro specimens.
- High-temperature stiffness ranking from asphalt binder tests (PG) did not match the ranking from loaded wheel track test for the evaluated asphalt binders and asphalt mixtures.

Recommendations

The objective of this study was to evaluate the durability and performance of OGFC mixtures containing various types of asphalt binder. Based on the study's findings, the following revision to specifications are recommended:

- Revise Section 501, "*Thin Asphalt Concrete Applications*" of the 2016 "Louisiana Standard Specifications for Roads and Bridges." Specifically, develop a rational method to ensure a stone-on-stone contact based on voids in coarse aggregate (VCA) requirement. Further, allow EA and PG 88-28 binders in OGFC construction. Moreover, incorporate Cantabro abrasion loss test to ensure durability.
- Construct a field project to evaluate performance of OGFC mixtures with 50%EA and PG 88-28 binders, as they exhibited the best performance for laboratory testing.
- Conduct a life-cycle assessment (LCA) to evaluate the environmental impact of the evaluated mixtures.

Acronyms, Abbreviations, and Symbols

Term	Description
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Content
ASTM	American Society for Testing and Materials
BBR	Bending Beam Rheometer
°C	degree(s) Celsius
CE	Cost-Effectiveness
cm	Centimeter
DOTD	Louisiana Department of Transportation and Development
DSR	Dynamic Shear Rheometer
°F	degree(s) Fahrenheit
FHWA	Federal Highway Administration
FS	Frequency Sweep
G_{ca}	Bulk specific gravity of the coarse aggregate
G_{mm}	Theoretical maximum specific gravity
G_{mb}	Bulk specific gravity of compacted mixture
HMA	Hot mix asphalt
JMF	Job mix formula
J_{nr}	Non-recoverable creep compliance
kPa	Kilopascal
LAS	Linear Amplitude Sweep
LTRC	Louisiana Transportation Research Center
LWT	Loaded Wheel Tracking
m	meter(s)
mm	millimeter(s)
NCAT	National Center for Asphalt Technology
NCHRP	National Cooperative Highway Research Program
NMAS	Nominal Maximum Aggregate Size
PAV	Pressure Aging Vessel

Term	**Description**
PG	Performance Grade
P_{ca}	Percentage of the coarse aggregate in the total mixture
RTFO	Rolling Thin-Film Oven
TSR	Tensile strength ratio
VCA	Voids in coarse aggregate

References

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Appendix

Appendix A:
Proposed Open-Graded Friction Course Mixture Design

Standard Specification for

Open-Graded Friction Course Mixture Design

AASHTO Designation: M X-25

Technical Subcommittee: X

American Association of State Highway and Transportation Officials
444 North Capitol Street N.W., Suite 249
Washington, D.C. 20001

Standard Specification for

Open-Graded Friction Course Mixture Design

AASHTO Designation: M X-25

1. SCOPE

1.1. This specification covers Open-Graded Friction Course (OGFC) mixture design using the Superpave Gyratory Compactor (SGC). The OGFC mixture design is based on the volumetric properties of the mixture in terms of air voids and stable aggregate structure. Long-term aging of OGFC mixtures followed R 30.

1.2. This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety concerns associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the application of regulatory limitations prior to use.

2. REFERENCED DOCUMENTS

2.1. *AASHTO Standards*:

- R 30, Mixture Conditioning of Hot Mix Asphalt (HMA)
- T 283, Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage
- T 401, Cantabro Abrasion Loss of Asphalt Mixture Specimens

2.2. *ASTM Standards*:

- C29/C29M, Test Method for Bulk Density ("Unit Weight") and Voids in Aggregate
- C127, Test Method for Relative Density (Specific Gravity) and Absorption of Coarse Aggregate
- C131, Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine
- C136, Test Method for Sieve Analysis of Fine and Coarse Aggregates
- C1252, Test Methods for Uncompacted Void Content of Fine Aggregate (as influenced by Particle Shape, Surface Texture, and Grading)
- D2041, Test Method for Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures
- D2419, Test Method for Sand Equivalent Value of Soils and Fine Aggregate
- D3203, Test Method for Percent Air Voids in Compacted Dense and Open Bituminous Paving Mixtures
- D4791, Test Method for Flat Particles, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregate
- D5821, Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate
- D6373, Specification for Performance Graded Asphalt Binder
- D6390, Test Method for Determination of Draindown Characteristics in Uncompacted Asphalt Mixtures
- D6752, Test Method for Bulk Specific Gravity and Density of Compacted Bituminous Mixtures Using Automatic Vacuum Sealing Method
- D6857, Test Method for Maximum Specific Gravity and Density of Bituminous Paving Mixtures Using Automatic Vacuum Sealing Method
- D6926, Practice for Preparation of Asphalt Mixture Specimens Using Marshall Apparatus

■ D6932, Guide for Materials and Construction of Open-graded Friction Course Plant Mixtures

3. TERMINOLOGY

3.1. *Definitions*:

3.1.1. *OGFC (Open-Graded Friction Course)*—special thin asphalt surface layer composed of a gap-graded structure with a high proportion of uniformly sized coarse aggregates. This structure creates interconnected air voids that allows for vertical and lateral drainage, reducing hydroplaning risk while also improving skid resistance, wet-weather visibility, and reducing tire–pavement noise.

3.1.2. *air voids (Va)*—the total volume of the air pockets between the coated aggregate particles throughout a compacted asphalt mixture, expressed as a percent of the total volume of the compacted specimen.

3.1.3. *voids in coarse aggregate (VCA)*—the volume in between the coarse aggregate particles, where this volume includes filler, fine aggregate, air voids, asphalt, and fiber, if used.

3.1.4. *nominal maximum size of aggregate*— the smallest sieve opening through which the entire amount of aggregate is permitted to pass.

3.1.5. *stabilizing additive*—polymer, crumb rubber, or fibers, used to minimize draindown of the asphalt during transport and placement of the OGFC.

4. SUMMARY OF METHOD

4.1. *Select Materials*—asphalt binder, aggregates, and additives that meet specification are selected.

4.2. *Select Optimum Aggregate Structure*—at least three trial aggregate gradings from the selected aggregate stockpiles are blended. The dry-rodded unit weight for the coarse aggregate for each trial grading is determined in accordance with Test Method ASTM C29/C29M. For each trial grading, an initial trial asphalt content of approximately 6.5% is selected, and at least two specimens are compacted using 50 gyrations of the Superpave Gyratory Compactor (SGC) (Test Method ASTM D6925) or other suitable compactor.

4.3. *Select Optimum Asphalt Content*—replicate specimens are compacted using 50 gyrations of a SGC or other suitable compactor at three asphalt contents. The design asphalt content is selected on the basis of satisfactory conformance with the requirements of Section 12.

4.4. *Evaluating Moisture Susceptibility*—the moisture susceptibility of the designed mixture shall be evaluated using the AASHTO T 324 test method for one-cycle freeze-thaw conditioned specimens in accordance with AASHTO T 283. If the mixture fails the selected moisture susceptibility requirement, it is suggested that appropriate modifiers such as liquid anti-strip, hydrated lime, or both are evaluated to meet the requirement.

5. SIGNIFICANCE AND USE

5.1. The procedure described in this practice is used to design OGFC mixtures that will provide a balanced performance by maintaining sufficient permeability for effective water drainage, thereby reducing the risks of hydroplaning and skidding while also ensuring durability and resistance to abrasion under high traffic volumes.

6. APPARATUS

6.1. *Oven*—an oven capable of maintaining any desired temperature setting from room temperature to 260°C to within ±3°C.

6.2. *Thermometers*—thermometers having a range from 60 to over 200°C and readable to 0.2°C.

6.3. *Balance*—a balance with a capacity of 2000 g readable to 0.1 g for determining the mass of asphalt binder.

6.4. *Superpave Gyratory Compactor (SGC)*—and molds conforming to AASHTO T 312.

6.5. *Mixer, bowls, spoon, spatula, etc.*

7. MATERIALS

7.1. Materials include aggregates, asphalt, ~~and~~ additives, ~~if any~~and other required materials.

7.1.1. *Selection of Coarse Aggregate*—coarse aggregate should have abrasion values of less than 25% in accordance with Test Method ASTM C131. Crushed gravel, if used, must have at least 90% particles with two faces and 95% particles with one face resulting from crushing in accordance with Test Method ASTM D5821. The percentage of flat and elongated particles should not exceed 10%, with a ratio of 3:1 in maximum to minimum dimension, respectively, in accordance with Test Method ASTM D4791.

7.1.2. *Selection of Fine Aggregate*—fine aggregate should have an uncompacted voids content of at least 40% when tested in accordance with Test Methods ASTM C1252, Method C. It is important that the aggregate be clean. The sand equivalent value of the fine aggregate passing the 2.36 mm [No. 8] sieve, according to Test Method ASTM D2419, should be at least 60% or greater. It is recommended that the material to be tested be separated on the 2.36 mm [No. 8] sieve because of the coarse grading of the aggregate.

7.1.3. *Asphalt Grade Selection*—asphalt grade selection is based on environment, traffic, and expected functional performance of the OGFC. Mixes with modified asphalt cements utilizing high Styrene-Butadiene-Styrene (PG 88-28) and epoxy modified asphalt have shown significant improvement in performance.

7.1.4. *Selection of Additives*—either a cellulose fiber or a mineral fiber may be used to minimize draindown. Typically, a dosage rate of 0.3% by mixture mass (or weight of total mix) is used, but the draindown target of 0.3% maximum should be the acceptance guideline for the dosage rate of the fiber stabilized additive.

8. HAZARDS

8.1. Use standard laboratory safety procedures required for handling the hot asphalt binder and required safety procedures when cleaning with solvents or degreasers.

9. PREPARATION OF OGFC COMPACTED SPECIMENS

9.1. *Number of Samples*—12 specimens are initially required: four samples at each of the three trial gradings. Each sample is mixed with the trial asphalt content (typically approximately 6.5%), and two of the four specimens for each trial aggregate grading are compacted. The remaining two specimens of each trial aggregate grading are then used to determine the theoretical maximum density.

9.2. *Selection of Aggregate Trial Gradings*—three trial gradings should be selected to be within the recommended master range of grading shown in Table 1. The three trial gradings should generally fall along the coarse and fine limits of the grading range, along with one falling in the middle. These trial gradings are obtained by adjusting the amount of fine and coarse aggregate in each blend.

9.3. *Selection of Trial Asphalt Content*—for each trial aggregate grading, an asphalt content between 6.0 and 6.5% should be initially selected based on the aggregates' bulk specific gravity.

9.4. *Preparation of Aggregates*—dry aggregates to a constant mass at 105 to 110°C [220 to 230°F] and separate the aggregates by dry-sieving into the desired size fractions.

9.5. Preparation of Mixtures:

9.5.1. A mechanical mixing apparatus shall be used.

9.5.2. For each test specimen, weigh into separate pans the amount of each size fraction required to produce a batch of aggregate that will result in a compacted specimen of the correct size. Mix the aggregate in each pan; place in an oven set to a temperature not exceeding the mixing temperature by more than approximately 28°C [80°F].

Heat the asphalt binder to the mixing temperature. The stabilizing additive or fiber, if used, should be added to the heated aggregate prior to the introduction of the asphalt. The stabilizing additive should be dry-mixed thoroughly with the heated aggregate.

9.5.3. *Compaction of Specimens*—laboratory samples of OGFC are short-term aged in accordance with AASHTO R 30, then compacted using 50 gyrations of the SGC or other compactor providing equivalent compacted density. Specimen diameter shall be 150 mm [6 in.] and nominal height shall be 115 mm [4.5 in.].

Table 1. Recommended Aggregate Trial Grading for OGFC (Percent Passing by Mass)

Sieve	Percent Passing
19.0 mm [3/4 in.]	100
12.5 mm [1/2 in.]	85 – 100
9.5 mm [3/8 in.]	50 – 70
4.75 mm [No. 4]	10 – 25
2.36 mm [No. 8]	5 – 10
0.075 mm [No. 200]	2 – 4

10. DETERMINATION OF VCA

10.1. An optimum aggregate grading is selected to ensure stone-on-stone contact. The stone skeleton is that portion of the total aggregate blend retained on the 4.75 mm [No. 4] sieve. The condition of stone-on-stone contact within an OGFC mixture is defined as the point at which the percent voids of the compacted mixture (VCA_{MIX}) is less than the VCA of the coarse aggregate in the dry-rodded test (VCA_{DRC}) in accordance with Test Method ASTM C29/C29M.

$$VCA_{DRC} = \frac{G_{CA}\gamma_w - \gamma_s}{G_{CA}\gamma_w} \times 100 \qquad (1)$$

G_{CA} = bulk specific gravity of the coarse aggregate (Test Method ASTM C127);
γ_s = bulk density of the coarse aggregate fraction in the dry-rodded condition (kg/m^3) (Test Method ASTM C29/ C29M); and
γ_w = density of water 998 kg/m^3 [62.3 lb/ft^3].

$$VCA_{MIX} = 100 - \left(\frac{G_{mb}}{G_{CA}} \times P_{CA}\right) \qquad (2)$$

$$V_a = 100 - \left(1 - \frac{G_{mb}}{G_{mm}}\right) \qquad (3)$$

G_{CA} = bulk specific gravity of the coarse aggregate (Test Method ASTM C127);
G_{mb} = bulk specific gravity of the compacted mixture;
G_{mm} = theoretical maximum density of the mixture; and
P_{CA} = percent coarse aggregate in the total mixture.

Of the three trial gradings evaluated, the one with the highest air voids (minimum acceptable is generally 18% as determined by vacuum sealing method by Test Method ASTM D6752) and a VCA_{MIX} equal to or less than that determined by the dry-rodded technique (VCA_{DRC}) is considered optimum and is selected as the desired grading for optimum aggregate stability and stone-on-stone contact.

11. SELECTION OF OPTIMUM ASPHALT BINDER CONTENT

11.1. After selecting the optimum aggregate gradation as outlined in Section 10, the following tests are required to evaluate the performance of the OGFC mixture. A draindown test shall be conducted to determine the maximum allowable asphalt binder content that ensures acceptable draindown during production and placement. The Cantabro abrasion loss test is necessary to determine the minimum binder content that provides sufficient cohesion between the asphalt binder and aggregates, thereby improving resistance to abrasion loss. The Hamburg Wheel-Track test shall be performed under wet conditions on specimens with no moisture conditioning and after one freeze-thaw cycle, to assess resistance to permanent deformation and moisture susceptibility, respectively. Lastly, the falling head water permeability test is required to ensure the mixture meets minimum drainability requirements and exhibits interconnected air voids.

11.2. The draindown test is conducted on a loose mixture at a temperature 10°C [50°F] higher than the anticipated production temperature using Test Method ASTM D6390. Fiber stabilizers are typically incorporated into the mixture at a rate of 0.2 to 0.5% of the total mixture mass to control draindown. The maximum permissible draindown should not exceed 0.3% by total mixture mass.

11.3. The Cantabro abrasion loss test is conducted using Test Method AASHTO T 401 on a compacted mixture conditioned for long-term aging according to AASHTO R 30. The average abrasion loss from the Cantabro test should not exceed 30%.

11.4. The Hamburg Wheel-Track test is conducted using Test Method AASHTO T 324 on a compacted mixture for two sets of specimens, one without moisture conditioning and another subjected to one freeze-thaw cycle according to AASHTO T 283. The average rut depth after 5,000 passes from the Hamburg Wheel-Track test should not exceed 6.0 mm and 8.0 mm for unconditioned and moisture conditioned specimens, respectively.

11.5. The falling head water permeability test is conducted using Test Method FM 5-565 T 401 on a compacted mixture. The coefficient of permeability should be equal to or greater than 100 m/day [328 ft./day].

11.6. *Adjusting Asphalt Content to Meet Requirements*—if the selected asphalt content did not meet the appropriate criteria, 11.2 – 11.5, the mixture may need to be further evaluated. The following are suggested mixture changes that may be helpful in making a mixture that meets the mix criteria. If the air voids are too low, the asphalt content should be reduced. If draindown values are greater than 0.3%, the amount of asphalt and/or type or amount of stabilizer shall be adjusted. If the abrasion loss on long-term aged specimens is greater than 30%, the asphalt content should be increased. If the mixture fails to meet the moisture susceptibility requirements, hydrated lime, liquid anti-strip, or both additives can be used. If these measures were ineffective, the aggregate source, the asphalt binder source, or both, can be changed to obtain better aggregate/asphalt compatibility.

12. REPORT

12.1. Report the following information:

12.1.1. Project type and number;

12.1.2. Aggregate source; asphalt source and grade; type and amount of stabilizing additive; and material quality characteristics;

12.1.3. Optimum aggregate gradation, along with the corresponding VCA and air voids results;

12.1.4. Draindown and Cantabro abrasion loss in percent, rut depth in mm, and coefficient of permeability in m/day;

12.1.5. Recommended job-mix formula for the OGFC.

13. KEYWORDS

13.1. Mixture design; open-graded friction course; fiber stabilizers; hydrated lime; anti-strip, thin surface treatments; durability and moisture susceptibility.