HIGH RECLAIMED ASPHALT PAVEMENT (RAP) ASPHALT MIXTURES FOR LOW VOLUME ROADS

A Thesis

by

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ABSTRACT

The inclusion of increased reclaimed asphalt pavement (RAP) quantities in the production of asphalt mixtures for low volume roads represents an environmental solution that manages to reduce waste and demand of fossil derivate products, and at the same time provides an opportunity for local agencies to optimize roadway construction and maintenance budgets.

The objective of this research project was to develop guidance regarding the methodology for the performance assessment of asphalt mixtures with elevated contents of RAP (i.e. 60% to 100%) and define their suitability to perform as surface layers of pavements for low volume roads (i.e. less than 750 vehicles per day). Performance was evaluated for eight hot and ten cold recycled mixtures in relation to common pavement distresses including moisture susceptibility, rutting resistance, intermediate temperature cracking, durability and stiffness by modified Lottman, Hamburg Wheel Track Test (HWTT), Semicircular Bending Beam (SCB) test, Cantabro abrasion loss test and Resilient Modulus (M_r), respectively. Compacted specimens were fabricated with RAP contents of 60, 80 and 100% employing two sources of RAP, limestone and granite virgin aggregates and two types of recycling agents per recycling methodology (hot and cold). One organic-based agent and one petroleum-based agent were selected for the hot recycling methodology and emulsified and foamed asphalt were selected for the cold recycling methodology.

The performance results demonstrated that the evaluation of rutting and moisture susceptibility through HWTT and Modified Lottman, respectively, is likely too severe for high RAP mixtures. With few exceptions, all the recycled mixtures displayed accelerated rutting.

However, hot recycled and cold recycled mixtures with emulsion exhibited capacity to perform as surface layers for low volume roads according to current standards based on requirements of tensile strength and moisture susceptibility. Cold recycled mixtures with foamed asphalt did not meet any distress threshold evaluated. Hot recycled mixtures including recycling agents exhibited improved cracking resistance in the SCB test, reduction in M_r and most of the cold recycled mixtures presented poor durability. Varied influence on the mixtures performance was observed for RAP content and recycling agent/additive type depending on factors such as virgin aggregate type, RAP source and recycling methodology. The inclusion of cracking resistance and durability thresholds in the design process of recycled mixtures was found effective at detecting better performing mixtures. In order to improve the assessment of rutting, it is recommended to investigate the use of dry tests such as asphalt pavement analyzer (APA). Likewise, further investigation is recommended with regard to the inclusion of active fillers (i.e., Portland cement or hydrated lime) to improve tensile strength and durability performance of cold recycled mixtures.

CONTRIBUTORS AND FUNDING SOURCES

This research project was supported by a thesis committee consisting of Dr. Amy Epps Martin (Chair), Dr. Jon Epps (Co-Chair) and Dr. Alan Freed (member). Supervision and guidance through the project tasks was provided by Dr. Edith Arámbula-Mercado.

Materials employed for the production of the recycled mixtures including recycled asphalt pavement (RAP), virgin aggregates and asphalt binders were provided by the Florida Department of Transportation (FDOT). Likewise, the laboratory tests for the performance evaluation of the recycled mixtures were conducted thanks to the economic contribution of FDOT.

Samples of the recycling agents and the emulsified asphalt were provided by manufactures in the industry.

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1. INTRODUCTION

In a society experiencing the effects of climate change and concerned with environmental stewardship, awareness for solutions that reduce energy consumption, carbon emissions and promote materials conservation has led to the use of recycled asphalt pavement (RAP) in the paving industry. With over four million miles of paved roads in the United States (Gaitan, 2012), the yearly pavement maintenance activities, including milling and resurfacing, generates RAP stockpiles of the order of 90 million tons (Hansen et al., 2017).

A large portion of the RAP that is generated is utilized in the production of hot mix asphalt (HMA), which seldom exceeds RAP contents of 20% to 25% (Al-Qadi et al., 2012). Other RAP applications include cold recycling operations for rehabilitation and construction of non-trafficked structures such as shoulders (Hansen et al., 2017). As a result, large quantities of RAP accumulate in urban and rural stockpiles. When RAP production rates exceed HMA plants capacity to process and effectively utilize it, it gives the contractors no other option than to haul the excess RAP material to landfills. Therefore, the inclusion of increased RAP quantities, (i.e. up to 60% and 100%) in the production of HMA and other asphalt mixtures for low volume roads represents an environmental solution that manages to reduce waste and demand of fossil derivate products, and at the same time provides an opportunity for local agencies to optimize roadway construction and maintenance budgets.

The objective of this research project was to develop guidance regarding the methodology for the performance assessment of asphalt mixtures with elevated contents of RAP (i.e. 60% to

100%) and define their suitability to perform as surface layers of pavements for low volume roads (i.e. less than 750 vehicles per day).

This project investigated the performance of asphalt mixtures including elevated quantities of RAP in order to determine their suitability to be used as pavement surface layers in low volume roads (i.e. less than 750 vehicles per day as defined by The Florida department of Transportation). To do so, recycled mixtures were fabricated through cold and hot recycling methodologies and their performance measured by means of various laboratory tests including: moisture susceptibility, intermediate temperature cracking, rutting resistance, durability and stiffness. Compacted specimens were fabricated with RAP contents of 60, 80 and 100% employing two sources of RAP, two virgin aggregate types and two recycling agents/additives per recycling methodology (cold and hot). Ultimately, the objective of this project was to provide guidance for the design and performance assessment of hot and cold high RAP mixtures.

Chapter 2 presents the current state-of–the-practice for hot and cold recycling methodologies regarding: mixture design procedures, laboratory performance testing and case studies. Chapter 3 presents the selection criteria and characterization procedures of the materials employed in the production of high RAP hot/cold recycled mixtures. Chapter 4 presents the design process of high RAP mixtures produced by means of both recycling methodologies and the results of laboratory performance evaluation. Chapter 5 presents further discussion about laboratory results and performance comparison between recycling methodologies. Chapter 6 presents general conclusions for each recycling methodology and a summary of relevant observations and conclusions regarding performance. Finally, Chapter 7 presents the list of bibliographic references consulted along this research project.

2. LITERATURE REVIEW

The benefits of pavement recycling practices have been widely studied and generally indicate economic and environmental benefits. Recent studies have determined that the use of RAP leads to the reduction of costs regarding the unit price of HMA and overall in rehabilitation and maintenance projects. The Turner–Fairbank Research Center of the Federal Highway Administration in (2010) determined that cost savings per ton of mixture on the order of 14% to 34% could be achieved using RAP contents between 20% to 50%. Likewise, a cost assessment conducted by Epps (2017) determined that cost savings from 35% to 45% can be expected in removal and pavement replacement through the implementation of cold recycling practices.

With regard to environmental effects of RAP usage, the benefits include conservation of natural resources, reduction in energy consumption and decrease in greenhouse gas (GHG) emissions. Research directed in (2005) by The Canadian Industry Program for Energy Conservation reported that 10% RAP content in HMA decreases energy consumption by 6%, while warm mix asphalt (WMA) technologies reach reductions of up to 4%. The study also determined that 50% RAP contents in HMA applications reduces energy consumption to the level required for the production of cold recycled mixtures.

According to Stroup-Gardiner and Wattenberg-Komas (2013), the main contribution to GHG emissions from the paving industry comes from burning fuel to dry aggregates, heat asphalt and HMA production in general. However, the current methodology for GHG emission quantification is limited in accounting for the effect of including RAP or any other recycled product in HMA. Modifications to the CO_2-emission factors defined by the Environmental Protection Agency (EPA) are suggested by the Stroup-Gardiner & Wattenberg-Komas (2013).

A sustainable pavement, according to Stroup-Gardiner and Wattenberg-Komas (2013), is defined as: “A pavement that minimizes environmental impacts through the reduction of energy consumption, natural resources and associated emissions while meeting all performance conditions and standards”. Therefore, minimum performance criteria must be met by high RAP mixtures employed in low volume roads, in order for the economic and environmental benefits to represent genuine advantages. However, the use of high RAP mixtures (i.e. 60 to 100%) presents several challenges. These mixtures perform very differently as compared to conventional HMA. High variability and severe asphalt binder aging are major concerns (Arambula, 2016). Li et al. (2004) found that the inclusion of RAP increased the variability in test results and the variability increased with decreasing temperatures. Moreover, research conducted by McDaniel et al. (2002) suggested that modifications to the asphalt binder grade required for a certain geographic area are necessary when RAP contents higher than 15% are employed. Similarly, Al-Qadi et al. (2009) showed that significant changes in the asphalt binder properties were observed for RAP binder contents greater than 20%. In addition, Lee et al. (2002), through testing of the asphalt binder in the dynamic shear rheometer, determined increasing binder stiffness and brittle behavior with increasing RAP binder. McDaniel et al. (2002) translated that into an improved mixture resistance to rutting but a consequent reduced resistance to fatigue cracking and increased susceptibility to thermal cracking. Hence, it is paramount to provide the recycled mixture with an asphalt binder that exhibits the rheological characteristics (i.e., stiffness and phase angle) as close to those of a virgin asphalt as possible.

Currently, hot and cold recycling are the major methodologies for pavement recycling. Both practices employ RAP resulting from rehabilitation and maintenance activities to produce a new asphalt mixture, but differ in whether or not energy, in the form of heat, is used in the process. Figure 1 displays a general classification of the pavement recycling methodologies used by the industry.

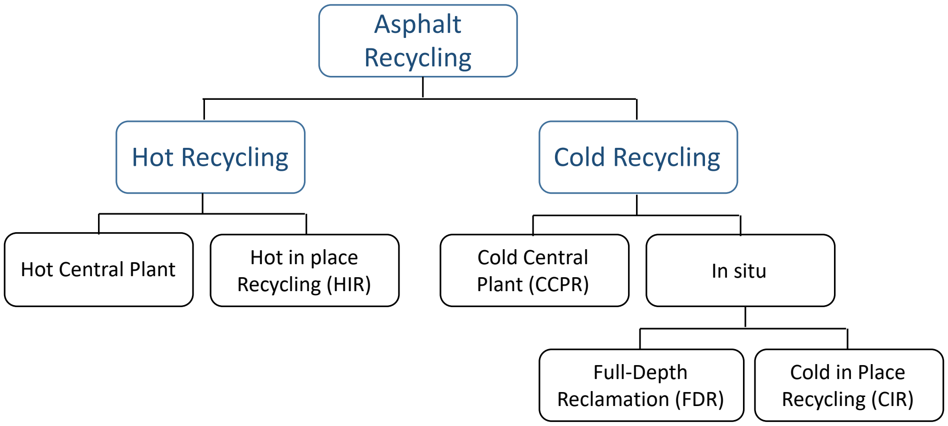


Figure 1. Classification of Pavement Recycling Methodologies

The following chapter describes the pavement recycling methodologies that this research project aims to evaluate for high RAP mixtures in terms of mixture design considerations and a few case studies.

2.1. HOT RECYCLING

Hot recycling is the most common method for pavement recycling used by contractors (Santucci, 2007) and combines RAP, virgin aggregate and asphalt binder in the presence of heat to produce a new HMA. The RAP might be obtained either from stockpiles and incorporated into the mixture at a central plant, or it can be milled and heated in-situ employing a recycling unit in a methodology known as hot in-place recycling (HIR). Present HIR equipment allows the reclamation of the top 1.5 to 2.0 in. (38.1 to 50.0 mm) of an existing pavement structure and it is considered a partial depth recycling methodology (ARRA, 2005).

As mentioned, one of the major challenges associated with high RAP usage is the inclusion of severely aged asphalt binders into the new HMA. Since the RAP material has already been in-service for a certain time, the asphalt binder in it has experienced some degree of oxidation and embrittlement. Therefore, the ability to resist cracking is a major concern for mixtures including considerable amounts of RAP. Furthermore, field performance data of mixtures including RAP reported by Anderson (2010), Bennert and Maher (2013), Hong et al. (2010), West et al. (2011) and Zhou et al. (2011) confirm their cracking susceptibility.

In order to address potential cracking problems in the recycled mixture, the hot recycling methodologies employ the addition into the mixture of softer asphalt binders or recycling agents. Soft binders are asphalt binders of high phase angle and low viscosity or stiffness. Anderson et al. (2010) and Hanson et al. (2010) characterized binder cracking susceptibility by means of a ΔT_c parameter, which is defined as the difference in the low temperatures required by binders to meet a maximum stiffness of 300 MPa and a minimum m-value of 0.3 in the bending beam rheometer (BBR).

Conversely, the objective of using recycling agents is to restore the rheological properties (i.e. reduce stiffness and increase phase angle) of the significantly aged asphalt binders that coat the aggregates in the RAP. Multiple types of recycling agents are available in the market, although they can be classified into two main categories: organic (i.e. vegetable oil derivatives) and petroleum derivatives (i.e. paraffinic oils, aromatic extracts and naphthenic oils) (Arambula, 2016). Tran et al. (2012) found improved binder and mixture fatigue performance with the inclusion of one recycling agent. Similarly, Booshehrian et al. (2013) reported improved performance of recycled mixtures in multiple laboratory tests when including three different recycling agents. Zhou and Im (2015) in a more detailed investigation found that the effectiveness in the performance improvement of the mixture depends on the recycling agent type and proportion.

2.1.1. Mixture Design Considerations

Most current mixture design procedures for HMA were developed when recycled materials and other additives were not popular components of asphalt mixtures, and therefore these conventional mixture design approaches do not account for the special characteristics of RAP and challenges associated with its use, frequently yielding mixtures of unsatisfactory cracking and/or raveling performance (Arambula, 2016).

In order to adapt to the increasing use of recycled materials (i.e. RAP) in the production of HMA, the Texas Department of Transportation (TxDOT) developed the balanced mixture design procedure in which volumetric factors, rutting and cracking resistance and moisture susceptibility are considered to determine an optimum binder content. Therefore, compacted specimens of a RAP + virgin aggregate blend, at a defined gradation, including a minimum of three asphalt binder contents are tested for rutting and cracking by means of the Overlay test (OT) and HWTT, respectively. The maximum binder content that prevents rutting and bleeding is then defined as the binder content that achieves 98% density. The optimum binder content is defined as the highest value between the binder content that satisfies a minimum of 300 cycles in the overlay test and a rut depth of ½ in. (12.5 mm) in the HWTT. To account for variables that influence the recycled mixture performance such as traffic, weather and pavement structure; TxDOT employs a simplified asphalt overlay design program called S-TxACOL. The software determines the number of overlay test cycles that guarantee adequate performance for a set number of months in-service.

No official national standard for the design of hot recycled mixtures is currently available. However, the general purpose is to select an optimum binder content that meets requirements of performance and/or volumetrics. Figure 2 presents the steps normally followed for the design of hot recycled mixtures.

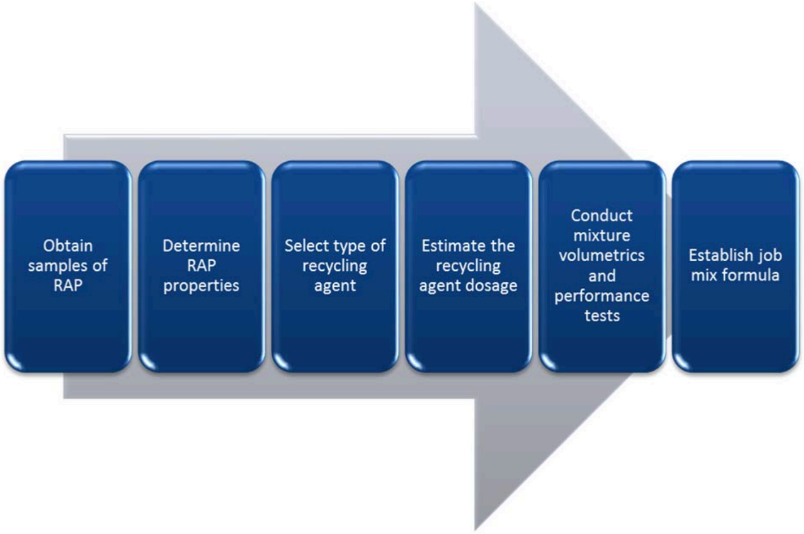


Figure 2. General Mixture Design Procedure Hot Recycling (Reprinted from Arambula, 2016)

2.1.1.1. *<u>RAP Characterization</u>*

As shown in Figure 2, the RAP source characterization is one of the key steps for the design of hot recycled mixtures. Particularly, the rheological characterization of the RAP binder is an important portion of this task and includes the determination of the stiffness and phase angle at high, intermediate and low temperatures. In comparison to most common virgin binders, recycled binders extracted and recovered from RAP sources require higher temperatures to develop stiffness lower than the minimum threshold of 1.0 kPa for the unaged high temperature performance grade (PG) classification. Temperatures from 80°C up to 100°C or more have been observed (Zhou et al. 2015).

Characterization of the RAP binder with aging is achieved by means of the Glover -Rowe (G-R) parameter. Measurement of stiffness (G*) and phase angle (δ) at a temperature of 59°F (15°C) and a frequency of 0.005 rad/s are used to compute the G-R parameter as defined by Glover et al. (2005) and shown in Equation 1 originally defined by Glover et al. (2005) and reformulated for practical use by Rowe (2011).

$$G - R = G^{*}\frac{\cos^{2}(\delta)}{\sin(\delta)}$$ **Equation 1**

When plotted in the Black space diagram (Figure 3), the changes in stiffness and phase angle, due to aging, shift the binder from the lower right to the upper left zone of the diagram. It is noteworthy that two different asphalt binders start at different locations in the diagram and will differ in the rate of translation from one zone to the other as aging occurs. Moreover, in order to help the identification of cracking due to brittle rheological behavior, a damage zone is set between G-R values of 180 and 600 kPa and correlates to low asphalt ductility values of 1.96 in. (50 mm) to 1.18 in. (30 mm), respectively, for field sections located in a PG 58-28 climate

(Kandhal 1977, Glover et al. 2005). These limits were previously related to surface raveling and cracking by Kandhal (1977).

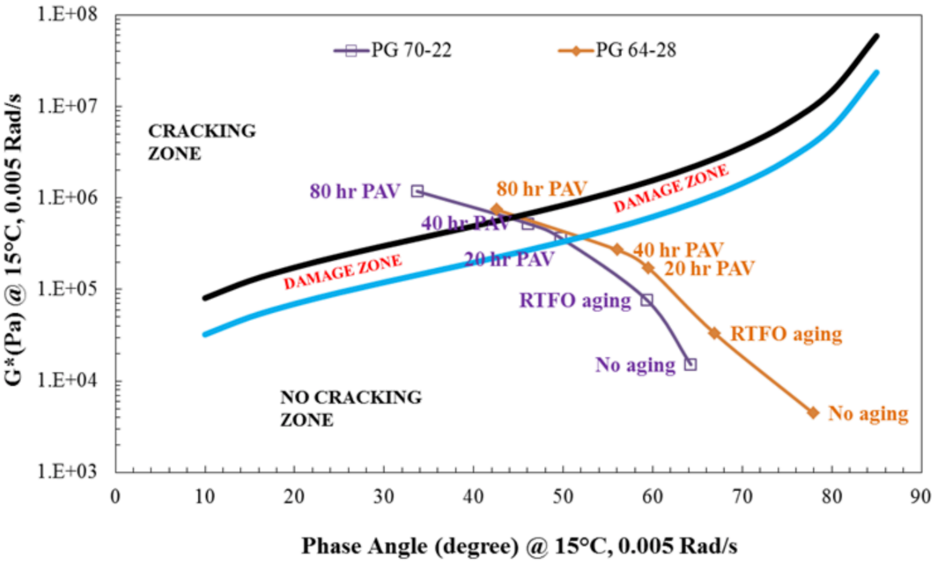


Figure 3. Black Space Diagram Example for Two Virgin Binders with Various Levels of Aging (Reprinted from Arambula, 2016)

In addition to rheological characterization (recycled binder grade), the RAP sources are normally evaluated to determine moisture content, gradation and binder content. RAP gradation and binder content are obtained via ignition oven method (AASHTO T 308), while the recycled binder grade is established after solvent extraction (AASHTO T 164 or ASTM D2172) and recovery (ASTM D1856 or ASTM D5404) of the recycled binder from the RAP material.

2.1.1.2. Recycling Agent Type and Dose

The addition to recycled mixtures of recycling agents aims to restore the rheological properties (i.e. reduce stiffness and increase phase angle) of the asphalt binder coating the aggregate particles in the RAP. Research conducted by Epps et al. (1980); Newcomb et al. (1984); Newcomb and Epps (1981), identified the purpose of adding recycling agents as: (a) restoring the aged binder by decreasing the stiffness for construction purposes and mixture performance in the field; (b) restoring the recycled mixture in terms of durability or resistance to cracking by increasing the phase angle of the binder; (c) providing sufficient additional binder to coat the recycled and virgin aggregates and (d) providing sufficient additional binder to satisfy mixture design requirements.

ASTM D4552: *Standard Practice for Classifying Hot-Mix Recycling Agents* categorizes the recycling agents in groups according to properties such as viscosity, flash point, specific gravity and others. According to ASTM D4552, the recycling agent selection depends on the amount and hardness of the RAP asphalt binder and recommends, for high RAP hot recycled asphalt mixture (i.e. no more than 30% virgin aggregate), recycling agents categorized in the groups: RA 1, RA 5, RA 25 and RA 75

Table 1 presents the classification of recycling agents according to the National Center for Asphalt Technology (NCAT, 2014) including commercially available products.

Table 1. Common Types of Recycling Agents (Reprinted from NCAT, 2014)

Category	Types	Description
Paraffinic Oils	Waste Engine Oil (WEO) Waste Engine Oil Bottoms (WEOB) Valero VP 165® Storbit®	Refined used lubricating oils
Aromatic Extracts	Hydrolene® Reclamite® Cyclogen L® Valero 130A®	Refined crude oil products with polar aromatic oil components
Napthenic Oils	SonneWarmix RJ™ Ergon HyPrene®	Engineered hydrocarbons for asphalt modification
Triglycerides & Fatty Acids	Waste Vegetable Oil Waste Vegetable Grease Brown Grease Oleic Acid	Derived from vegetable oils
Tall Oils	Sylvaroad™ RP1000 Hydrogreen®	Paper industry by-products. Same chemical family as liquid antistrip agents and emulsifiers

The effects of including recycling agents can be assessed through several methods, all of which aim to determine the change in the RAP's rheology or performance after exposing it to different recycling agent doses. The assessment can be conducted either at small-scale with binder blends or at large-scale with recycled mixtures. The most common methods at small-scale test the rheological properties after: (a) extracting the RAP binder and blending it with different recycling agent doses and (b) after adding different doses of recycling agent to the RAP and extracting the treated RAP binder. The large-scale methods test the performance of recycled HMA treated with recycling agents when: (a) the recycling agent is added to the virgin asphalt prior to mixing as it is commonly done by the industry or (b) the recycling agent is added to the RAP in a marination process before mixing.

Blending charts such as the one presented in Figure 4 are frequently used in order to estimate the high-temperature PG (PGH) of the "RAP binder + Virgin binder" blend (PGH_{Blend}). In this chart the horizontal axis is the RAP content expressed in terms of replacement of the virgin binder and the high-temperature PG of the RAP binder and virgin binder are plotted in the primary and secondary y-axis, respectively. The same procedure can also be optionally followed for the low-temperature PG.

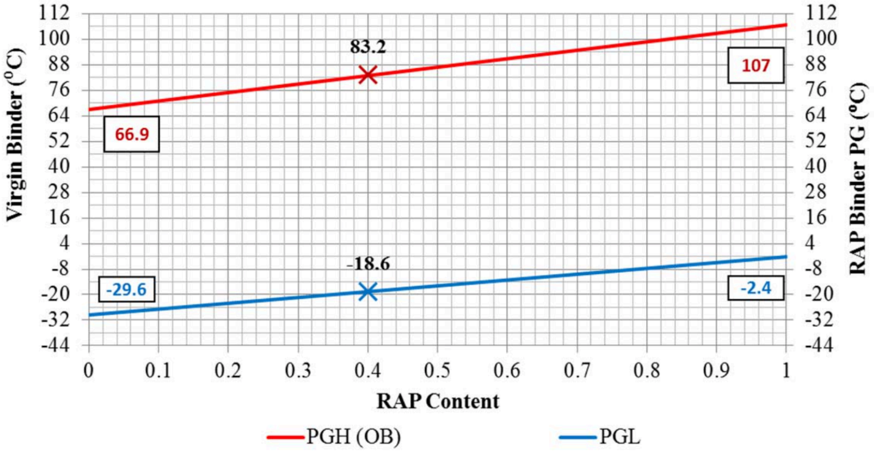


Figure 4. Example of a RAP Blending Chart (Reprinted from Arambula, 2016)

After studying multiple sources and grades of virgin binders, recycled materials and types of recycling agents; ongoing NCHRP project 09-58 (Kaseer et al., 2018) developed a methodology (Equation 2) to estimate the initial recycling agent dose, in which the PGH of the blend without the addition of recycling agents (PGH_{Blend}) and the PGH for the specific location where the recycled mixture will be used (PGH_{Target}) are required.

$$Recycling\ Agent\ Dosage = \frac{PGH_{Blend} - PGH_{Target}}{1.7} \qquad \textbf{Equation 2}$$

In order to verify the initial dose of the recycling agent, blends of RAP binder and virgin binder are prepared with 0, 2, 5 and 10% recycling agent and the high and low temperature PG are determined for each blend. To fully validate the dose, the PGH is used to determine the maximum dose allowed to be incorporated without compromising the rutting and cracking resistance (i.e. Unaged G*/sin(δ)>1.0 kPa). Hence, the dose is selected to match the PGH_{Blend} with PGH_{Target}. Figure 5 presents an example.

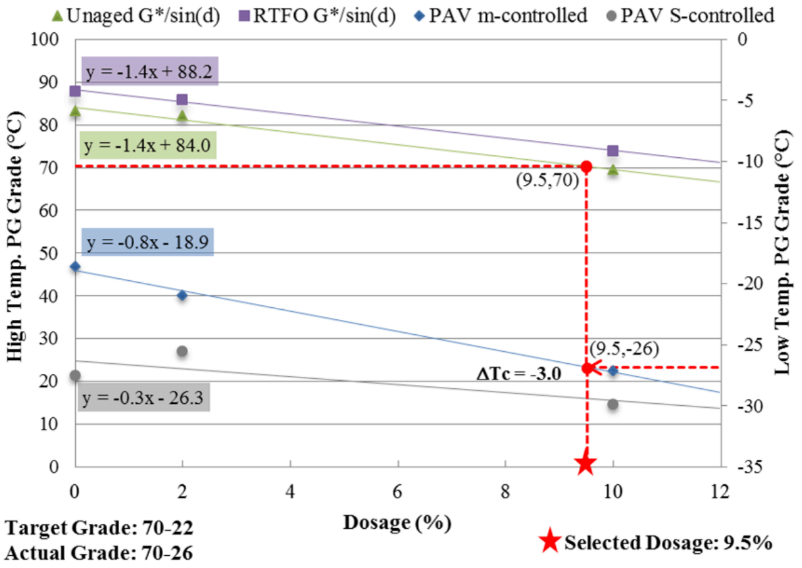


Figure 5. Example of Recycling Agent Dose Verification (Reprinted from Arambula, 2016)

2.1.1.3. *<u>Performance Testing</u>*

The performance of hot recycled mixtures has been proven to be adequately characterized by means of traditional HMA laboratory tests including resilient and dynamic modulus, indirect tensile strength, flexural fatigue, repeated shear load, flow number, Hamburg wheel track, overlay cracking, semi-circular bending beam and others (Arambula, 2016). Conventional and standardized laboratory tests procedures used to characterize HMA can also be used to assess the performance of high RAP hot recycled asphalt mixtures.

2.1.2. Case Studies

This section presents information regarding mixture design methods, materials, construction practices and observed field performance of past field projects where hot recycled mixtures were implemented. These past experiences highlight the characteristics, benefits and challenges associated with the recycling methodology.

2.1.2.1. Florida CR 315 and SR 19

In 2001, FDOT rehabilitated two projects utilizing HIR (Sholar et al. 2002). The first project was located on CR 315 in Putnam County between SR 100 and SR 20 with a total length of 7.58 miles. The second project was located on SR 19 between SR 40 and the town of Pittman in Lake County with a total length of 9.73 miles. Since at the time these projects were being considered industry representatives were concerned about the method employed to reclaim the surface layer of the existing pavement, FDOT decided to evaluate in-place milling on CR 315 and scarification on SR 19.

The recycled mixture for both projects was designed following Marshall criteria of 50 blows per sample face. For CR 315, 2.0 percent by weight of mixture of Type S-I structural mixture and 0.04 percent by weight of binder of AES-300RP recycled agent were added. For the SR-19 project 8.0 percent by weight of mixture of S-1-B South Florida limestone and 1.5 percent by weight of binder of Reclamite recycling agent were added. The reclamation depth on both projects was 1.5 in. (38.1 mm) and compaction done using a steel-wheeled vibratory roller and/or a rubber-tire roller to a target density of 92 percent. The average densities measured via field cores after construction were 92.6 percent for CR 315 and 94.4 percent for SR 19.

In order to evaluate the bond strength between the recycled mixture and the underlying layer, researchers employed a shear device on field cores obtained from various locations throughout the project and also on cores obtained on a nearby section where conventional milling and virgin HMA resurfacing was used. The results for CR 315 showed no differences between the two sets of field cores. No comparison was performed for CS 19, but the bond strength results for the recycled mixture were higher than the ones obtained for the recycled mixture on CR 315. Other performance indicators measured after construction such as friction and ride quality were also acceptable.

However, about two weeks after completing the construction of CR 315, cracking and delamination became apparent as shown in Figure 6. The distress progressed in extent and number of affected locations until about 50 percent of the project was affected. Researchers conducted a forensic evaluation and determined that a combination of several factors could have caused the failure, including excess dust generated during milling, higher dust content and lower binder content in the mixture, low mixture temperature during construction and variable layer thickness. Due to the extent and severity of the distress, the entire project was milled and resurfaced in 2002 using conventional HMA.

Figure 6. Cracking and Delamination on CR 315 (Reprinted from Sholar et al., 2002)

Although not all the parameters measured during construction of CS 19 met specifications (i.e., high Marshall flow, low air voids and low mixture temperature during construction), performance in terms of rutting, cracking, friction and ride quality for that project were adequate.

2.1.2.2. *Florida SR 471*

In 2002, FDOT employed HIR to rehabilitate a five-mile section of SR 471 south of Tarrytown in Sumter County that had severe cracking (cracking rating of 4.5 out of 10). This road was two-lanes wide with paved shoulders and an annual ADT of 2,800 vehicles. During construction, the top 2.0 in. (50 mm) were removed and combined with clean concrete sand to increase the air void content and an oil-based recycling agent named Sundex 540T. Marshall type S-III HMA was also added to correct the cross-slope as needed. The mixture designs for the northbound and southbound directions were slightly different given the in-situ properties (Sholar et al. 2004).

The roadway rehabilitation was completed in 22 calendar days, after which the produced surface presented a ride quality equivalent to a conventional HMA. Specifications requirements for rideability and friction were met as well. Additionally, the designers reported the mixture fulfillment of laboratory properties requirements for air voids, density and viscosity. However, after 6 months of service life, the surface began to present incipient rutting, apparently in the same locations where rutting was present prior to the rehabilitation. After one year of service, rutting exceeded the contract defined warranty threshold of 0.25 in. (6.35 mm).

Since this project required a three-year performance warranty by the contractor, a forensic investigation on the failed layer was conducted in a separate research project in an effort to determine the cause of rutting (Hammons and Greene 2006). Researchers found, based on falling weight deflectometer (FWD) results, a relevant composite pavement stiffness difference between the lots that exhibited high rutting and those who did not. Likewise, tests performed on reclaimed cores indicated that compaction due to traffic loading was a contributing factor to the observed rutting.

2.1.2.3. New Hampshire I-93

In 2015, the New Hampshire DOT (NHDOT) sponsored a research project to evaluate the performance of high RAP pavements (up to 40 percent) through a series of field assessments and laboratory tests. The study corresponds to the second phase of an NHDOT-sponsored project and was conducted on six sections of Interstate Highway 93 (I-93) in Woodstock and Lincoln. The test sections were constructed in 2011 and, by the time of the study, they presented about 3.5 years in-service (Daniel et al. 2015).

The test sections were divided in two categories according to the binder grade. PG 58-28 binder and RAP contents of 0, 15 and 25 percent were part of the first group, while PG 52-34 binder and RAP contents of 25, 30 and 40 percent were part of the second group. Accordingly, six different mixtures were produced using two different binder grades and RAP contents with a nominal maximum aggregate size of 0.5 in. (12.5 mm) and an optimum binder content of 5.8%.

The laboratory investigation included measurements of dynamic modulus, fatigue resistance, flow number and rutting/moisture susceptibility via Hamburg wheel tracking test on field cores, plant mixed plant compacted (PMPC) specimens, plant mixed laboratory compacted (PMLC) specimens and laboratory mixed laboratory compacted (LMLC) specimens. All laboratory specimens were compacted using the Superpave gyratory compactor (SGC) to a target air void content of 6 percent. Ten field cores were extracted per test section.

The laboratory results showed that mixtures with PG 58-28 binder were stiffer than those with the PG 52-34 binder and that the stiffness of the mixture increased with added RAP content, as expected. The binder grade had a bigger influence on stiffness than the increasing RAP content. The rutting /moisture susceptibility also showed expected trends with increasing rutting resistance for higher RAP contents. Regardless of binder grade or RAP content, all mixtures exhibited adequate rutting and moisture susceptibility. Within each mixture type, all specimen types followed similar trends except for the PMLC specimens. The observed differences are likely due to the re-heating process of the loose mixture necessary for compaction.

Field performance evaluation of the section via surface distress survey after 3.5 years in-service showed better thermal and fatigue cracking performance for the mixtures with the PG 58-28 binder, whereas no difference was observed with increasing RAP content. Therefore, researchers concluded that the use of a softer binder (i.e., PG 52-34) did not have a significant impact on field performance.

2.2. COLD RECYCLING

Cold methodologies for pavement recycling do not require energy in the form of heat to produce new asphalt mixtures. Therefore, RAP, virgin aggregates, additives and, if necessary, water and active fillers such as Portland cement or lime, are mixed at ambient or relatively low temperatures. The cold recycling methodologies usually incorporate bitumen-based products, such as emulsions or foamed asphalt, as additives (ARRA, 2005). The additives act as bonding material and, in contrast to the recycling agents used in hot recycling, they do not restore the RAP binder rheological properties. In certain cases, chemical additives or active filler (i.e. hydraulic cement or hydrated lime) are included in the mixture to improve the strength gain and stripping performance (ARRA, 2015).

In-place and central-plant recycling alternatives are the most common cold recycling practices employed for pavement rehabilitation and maintenance activities. The cold in-place recycling (CIR) methodology typically reclaims 2 up to 5 in. (50 to 127 mm) of the existing pavement surface layer employing a moving “train” of single or multiple units that mill, crush, screen, mix and pave. In this process 100% of the on-site RAP is utilized. For the cold central plant recycling (CCPR) methodology, the RAP often comes from an existing stockpile or is milled beforehand to then be mixed in a stationary mixing unit on site (ARRA, 2005). At this point, CCPR allows the contractor to select the RAP content to include in the final mixture.

On low volume roads, where the CIR/CCPR layer functions as a wearing course, a seal coat is recommended to prevent raveling and moisture damage (ARRA, 2005).

2.2.1. Mixture Design Considerations

For the design of cold recycled mixtures, it must be considered whether or not the asphalt coating the aggregate particles in the RAP contributes to the overall cementation of the mixture. Since the temperature of the virgin aggregate and RAP material during the production of cold recycled mixtures rarely reaches values over 212°F (100°C), the degree to which the binder in the RAP softens in order to act as the mixture's cement is a concern. Therefore, three possible outcomes are identified: (1) the RAP will act as a "black rock" leading it to behave as ordinary aggregate particles, (2) all the asphalt binder in the mixture will soften through the addition of a recycling agent or (3) partial softening of the RAP binder is achieved.

Cold recycled mixtures generally exhibit base-type characteristics, due to which most of the procedures currently available for their design employ base materials guidelines such as requirements of gradation and moisture content (MC) at mixing and compacting.

A report by AASHTO-AGC-ARTBA on cold recycling of asphalt pavements is one of the first standardized mixture design procedures for these materials and is based on Marshal stability and compaction with Hveem equipment (AASHTO 1998). Table 2 compares current DOT standards and special provisions for the design of cold recycled mixtures stabilized with emulsified or foamed asphalt. Standards such as the ones presented in Figure 3 usually specify or provide guidelines regarding the type of recycling agent, water content at mixing and compacting, additives and performance requirements.

With respect to moisture content at mixing and compacting, most agencies provide intervals for the optimum moisture content, which usually range between 1 and 3% (Table 2). Other agencies quote standards for the determination of maximum density-moisture relationships.

Moreover, specimen compaction is predominantly specified through Superpave gyratory compactor at 30 to 35 gyrations, although some agencies allow 75-blow compaction per Marshal method. The specimens' size alternates between 4 to 6 in. (100 to 150 mm) diameter depending on the property and test specified. Likewise, variability is observed in the protocols for curing of compacted specimens. The agencies' curing methodologies state temperatures between 104 to 140^{o}F (40 to 60^{o}C) and fixed times from 16 to 48 hours or until constant mass is achieved (i.e. 0.05% mass change in two hours).

To determine the optimum binder content, the agencies agree about testing at least three binder contents. The set of tests and thresholds to be met vary but the general concept of minimum tensile strength is the same. Some variations include, for example, moisture conditioning of samples through modified Lottman (i.e vacuum saturation and freeze thaw cycle) as required by ARRA CR 201, retained tensile strength (TSR) thresholds ranging between 55% and 70% (Mississippi DOT Special Provision 907-425-1 and Illinois DOT, respectively) and additional performance thresholds as the ones used by Iowa DOT (i.e. Thermal cracking and Raveling Test).

Figure 7 presents the general steps followed by current guideless for the design of cold recycled mixtures according to ARRA (2015):

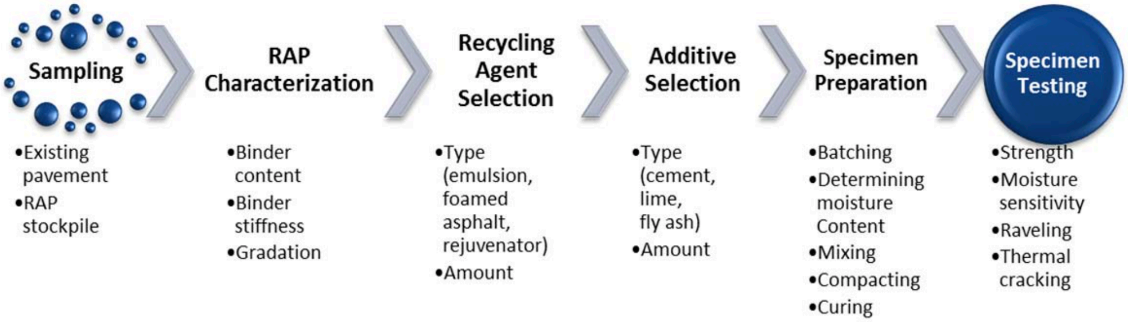


Figure 7. Cold Recycling Mixture Design Steps (Reprinted from Arambula, 2016)

Table 2. Comparison of CIR Mixture Design Standards and Special Provision

State	ARRA (2016)	Wirtgen Group (2012)	Wirtgen Group (2012)	California (2005)	Texas (Draft)	Texas (2015)	Colorado (2014)	Illinois (2012)	Kansas (2014)	Iowa (2016)
Specification	CR201	Cold-Recycling Manual	Cold-Recycling Manual	LP-8	Trial Specification	S.S. 3017	CP-L 5111	S.P. LR 400-5	C.M. Part V- 5.3.4	I.M. 504 App. B
Stabilizing Agent	Emulsion	Emulsion	Foamed Asphalt	Emulsion	Emulsion	Foamed Asphalt	Emulsion	Emulsion	Emulsion	Emulsion
Mixture & Compaction MC	1.5 to 3.0%	OFC (Optimal Fluid Content AASHTO T-180)	75% OMC (Optimal Moisture Content)	1.5 to 2.5%	Optimum Moisture Content Tex-113-E	Optimum Moisture Content (Tex-113-E)	2%MC	MC needed for emulsion dispersion	1.5 to 2.5%	1.5% MC
Compaction	75 blows per side by a Marshall 30-Gyrations SGC	Modified Marshall (75 blow per face)	Modified Marshall (75 blow per face)	75-blow Marshall or 30- gyration SGC	Minimum of 50 and a maximum of 60 blow of a10-lb. rammer.	Compact test specimens (Tex-206-F)	30-Gyrations SGC	30-gyration SGC	30-gyration SGC	30-gyration SGC
Curing	60 ± 1 °C to constant weight for at least 16 hours but not more than 48 hours	Oven at 40°C to constant mas (72h)	Oven at 40°C to constant mas (72h)	60 °C to constant mass (in 16 to 48 h)	72 hr. at 40°C before testing	Oven dry test specimens at 40°C for a minimum of 72 hr.	140ºF (60ºC) to constant mass	Not Specified	60 °C to constant mass (in 16 to 48 h)	60 °C for 48 h
Density	Bulk Specific Gravity of Compacted, Cured Specimens AASHTO T 166 (ASTM D2726) Maximum Theoretical Specific Gravity AASHTO T 209 (ASTM D2041)	Bulk Specific Gravity	Bulk Specific Gravity	Maximum Theoretical Specific Gravity AASHTO T 209 (ASTM D2041) Bulk Specific Gravity of Compacted, Cured Specimens AASHTO T 166 (ASTM D2726)	Maximum Dry Density (DA) Tex-113-E	Maximum Density determined (Tex-113-E)	Maximum specific gravity of the sample according to CP 51 (AASHTO T 209 or ASTM D2041)	Bulk Specific Gravity (Density), ASTM D 6752 or ASTM D 2726 Rice (Maximum Theoretical) Specific Gravity, ASTM D 2041	Maximum Theoretical Specific Gravity AASHTO T 209 (ASTM D2041) Bulk Specific Gravity of Compacted, Cured Specimens AASHTO T 166 (ASTM D2726)	Bulk specific gravity ASTM D 2726
Design Binder Content Selection Test	1. Gradation of Un-extracted RAP. AASHTO T 11b & T 27 (ASTM C117b & C136) 2. Indirect Tensile Strength AASHTO T 283 (ASTM D4867) 3.Marshall Stability AASHTO T 245 (ASTM D6927) 4.Tensile Strength Ratio/Retained Marshall Stability based on Moisture Conditioning AASHTO T 283 (ASTM D4867)	1. Indirect Tensile Strength Dry. 2. Indirect Tensile Strength Soak. 3. Tensile Strength Retained (TSR)	1. Indirect Tensile Strength Dry. 2. Indirect Tensile Strength Soak. 3. Tensile Strength Retained (TSR)	1. Marshall Stability. AASHTO T245 2. Retained Marshal Stability. AASHTO T245 3. Raveling Test. ASTM D7196	1. Unconfined Compressive Strength (UCS), Tex-117-E Part II 2. Indirect Tensile Strength (IDTS), Tex-226-F1. 3. Retained Unconfined Compressive Strength, Tex-117-E	1. Indirect Tensile Strength (IDT), psi Tex 226-F 2. Moisture Conditioned IDT, psi Tex 226-F 3. Moisture Conditioned Unconfined Compressive Strength, psi Tex 117-E, Part II	HVEEM Stability: CP-L 5106 (T246) Resistance To Moisture Induced Damage – Lottman Testing: CP-L 5109	1. Indicator Marshall Stability, ASTM D 1559. 2. Retained Stability AASHTO T245 3. Raveling Test. ASTM D7196	1. Marshall stability, KT-14. 2. Retained stability based on cured stability. 3. Raveling Test, KTMR-38	1. Marshall Stability ASTM D 1559 Part 5 2. Retained Stability AASHTO T245 3. Thermal Cracking FHWA LTPPBind software for 50% reliability at 3 in. below the pavement surface 4. Raveling Test. ASTM D7196
Test criteria	1. 1.25 in. (. (31.5-mm) Maximum Per Table 3 2. Minimum 45 psi (310 kPa) 3. Minimum 1,250 lbs. (5,560 N) g 4. Minimum 0.70	Bituminous Stabilized Material (BMS) Class 1- More than 3 million ESALs 1. ITS Dry > 225 kPa. 2. ITS Soak >100 kPa.	Bituminous Stabilized Material (BMS) Class 1- More than 3 million ESALs 1. ITS Dry > 225 kPa. 2. ITS Soak >100 kPa.	1. 5.56 kN min at 40 °C 2. 70% min at 40°C after V.S. and 24 h soak 3. 2% max, 20-gyr,cured at 21 °C for 4 h	1. 120 psi min. 2. 50 psi min. 3. 80% min.	1. Min 45 psi. 2. Min 30 psi. 3. Min 120 psi.	Highest emulsion content providing the highest stability, with the highest TSR (Tensile Strength Retained) from Lottman testing, and voids between 6% and 12% in the compacted sample	1. 1250 lb (567 kg) minimum. 2. 70% minimum. 3. 2% maximum at 50 ° F (10ºC)	1. 5.56 kN, min at 104o F (40o C) 2. 70% min, at Vacuum sat. of 55 to 75 percent, water bath 770 F (25o C) @ 23 hours, last hour at 1040 F (400 C) water bath. 3. 2 % max at ambient temperature.	1. 1000 lb min. at 100ºF (40ºC) after 2 hour temperature conditioning in a forced draft oven 2. 70% min at saturate to 55% to 75%, soak in a 75ºF (25ºC) water bath for 23 hours, followed by a 1 hour soak at 100ºF (40ºC) 3. -20ºC max. 4. 2% Max.

2.2.1.1. *RAP Characterization*

At a minimum, the RAP sources are evaluated to determine moisture content, gradation and binder content. Traditional sieving and ignition oven tests may be employed. Considering the scenario where the RAP material acts as a “black rock”, the rheological characterization of the binder coating the aggregate particles in the RAP is not very common for cold recycling technology. However, for other scenarios where the RAP binder softens to a certain level, extraction and recovery of the RAP binder for establishing the PG is recommended.

2.2.1.2. *Additives*

Emulsified and foamed asphalt are the most popular additives for pavement recycling through cold recycling methodologies. When added to the mixture, these two types of agents act as bonding material and, in contrast to the ones used in hot recycling, they do not attempt to restore the RAP binder rheological properties.

The emulsified asphalt, also known as emulsion, is the mixture of asphalt binder, water and emulsifier. Thanks to the latter, the asphalt binder disperses through the water phase and allows it to be fluid at ambient temperatures, which stands as its principal advantage with respect to ordinary binders. Emulsions may present a negative (anionic) or a positive (cationic) electric charge and a low to high rate of setting according to the properties of the emulsifier. Similarly, foamed asphalt is the mixture of hot asphalt binder, water and air. However, the mixture with water occurs simultaneously as the binder is sprayed into to aggregate blend (i.e. RAP and virgin aggregate blend). The contact of small quantities of water at ambient temperature with heated asphalt induces the formation of bubbles in the binder (i.e. foaming) and leads to the reduction of the binder’s viscosity. This way, the dispersion and mixing with the aggregates is assisted.

Initial doses for optimum asphalt content may be estimated employing monographs such as the one shown in Figure 8. This procedure requires as input the percent passing No. 40 and No. 200 sieves. Nevertheless, further verification of the output is recommended.

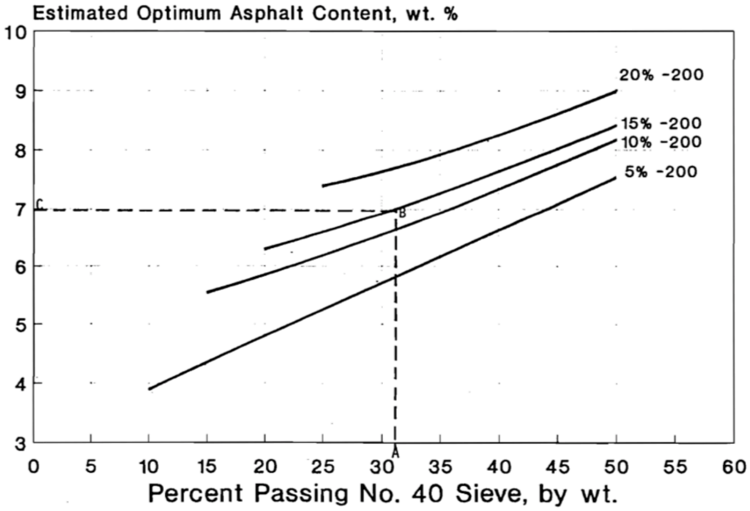


Figure 8. Nomograph for estimating the optimum Asphalt Content for Cold Recycled Mixtures (Reprinted from Estakhri, 1993)

2.2.1.3. Active Fillers

Active fillers or fillers are fine grained pozzolanic materials that chemically react with other mixture components, generally water, in order to boost the tensile strength and increase the rutting and moisture resistance. Popular fillers include lime, hydraulic cement in dry or slurry form and in very few cases fly ash. However, excessive doses of additives tend to result in brittle behavior and make the mixture more susceptible to cracking (ARRA, 2015).

Different practices are followed by state agencies regarding the use of additives. Some DOTs allow their use, but do not specify the type or dose (i.e. Iowa I.M. 504 App. B). Others restrict the allowed additives to cement and lime, but do not provide guidance on dose (i.e. New York, Cross et al. 2010). Agencies such as Mississippi (Special Provision 907-425-1) define 1% hydrated lime by mass for every cold recycled mixture, whereas the standard ARRA CR 201 and Illinois DOT (Special Provision defines LR 400-5) define additives' dose with respect to a set value for the ratio of residual asphalt to hydraulic cement.

A bibliographic search conducted by Cox et al. (2015) summaries the results of at least nine different references in order to compare the performance of cold recycled mixtures stabilized with emulsion and including hydraulic cement. The results determined that the additive's inclusion in the recycled mixtures improved the resilient modulus, moisture susceptibility and rutting resistance. However, a negative impact with respect to fatigue and thermal cracking resistance was also identified.

2.2.1.4. Mixture and Specimen Preparation

The production of cold recycled mixtures starts with the combination of RAP and virgin aggregate in proportions found to meet gradation requirements. Although cold recycling projects of 100% RAP are usual, it is a common practice to introduce virgin aggregate in the mixture in order to control the gradation and improve stability. The mixing of aggregate blend, additives and moisture is done at ambient temperature. In the case of foamed asphalt, depending on the binder foaming properties and grade, the binder is heated up to a temperature from 320 to 374^{o}F (160 to 190^{o}C). Emulsified and foamed asphalt contents typically range from 0.5-4.0% and 0.5-3.0% by weight of RAP, respectively (Arambula, 2016).

Multiple methods have been proposed and investigated for determining the optimum moisture content (OMC) of recycled mixtures. Conventional soil methodologies such as Proctor has been identified to determine considerably high moisture contents on the order of 8% and above (Cox et al., 2015). Marshall design is recommended by some DOT agencies since density and strength information is provided. Kim et al. in (2007) attributed the difficulty of determining sound values of OMC for cold in-place recycled (CIR) aggregate blends to RAP coarseness and lack of fines.

Some of the current DOT standards and special provisions for CIR provide OMC intervals which usually range between 1 and 3% (ARRA CR 201, CalTrans LP-8, Colorado DOT CP-L 5111, Kansas DOT C.M. Part V- 5.3.4). Mamlouk and Ayoub (1983), Scholz et al.(1991), Khosla and Bienvenu (1996) and Kim et al. (2011) fabricated cold recycled mixtures stabilized with emulsion employing arbitrary fixed values of moisture ranging from 1% to 5%. Babei and Walter (1989) and Kim (2011) in a more recent investigation defined a MC limit of 4% in order to achieve proper compaction. Figure 31 presents a histogram put together by Cox et al. (2015) displaying the mixing MCs employed in 43 CIR projects. The results show an average MC of 3.5% and a mode of 4% with a frequency around 40.

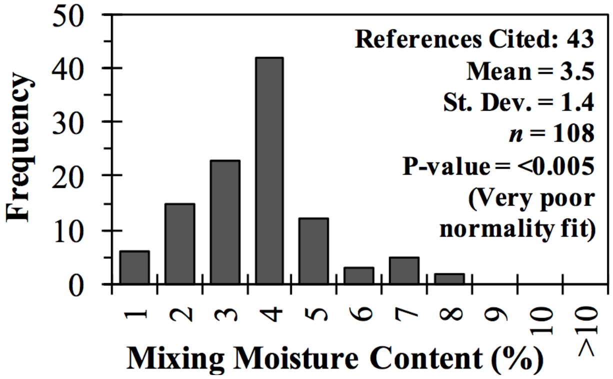


Figure 9. Cold In-Place Mixing Moisture Contents (Reprinted from Cox et al., 2015)

Compaction of specimens is conducted at ambient temperature and generally it is specified by state DOTs to a certain number of gyrations (i.e. 30 to 35) in the Superpave gyratory compactor (Table 2). Other agencies such as California, allow compaction by means of Marshall methodology. In general, there is consistency throughout standards and special provision about the compaction procedures of cold recycled specimens.

After compacting, and depending on the additive, curing time might be necessary to allow the mixture to set and the moisture to evaporate. In the field this is achieved by opening the road to traffic after a certain time, usually days, after compacting. In the laboratory, curing is achieved by placing compacted specimens in a forced draft oven at temperatures ranging from 104 to 140^{o}F (40 to 60^{o}C) for a minimum of 16 to a maximum of 48 hours. Most current guidelines for cold mixture design limit curing until 0.05% mass loss is achieved in a time period of two hours (i.e. constant mass).

2.2.1.5. *Performance Testing*

After compacting the specimens, performance assessment is typically conducted to evaluate moisture susceptibility of the recycled mixture. However, additional tests for the evaluation of durability, stiffness and rutting resistance are recommended. Most current guidelines for cold mixture design include thresholds for indirect tensile strength and some sort of moisture susceptibility (Table 2). Standard tests include modified Lottman and Retained Marshall stability. Other agencies such as TxDOT, define for CIR mixtures a minimum of 5,000 to 15,000 load cycles before reaching rut depths greater than half an inch in the HWTT. Iowa DOT requires thermal cracking and raveling verification.

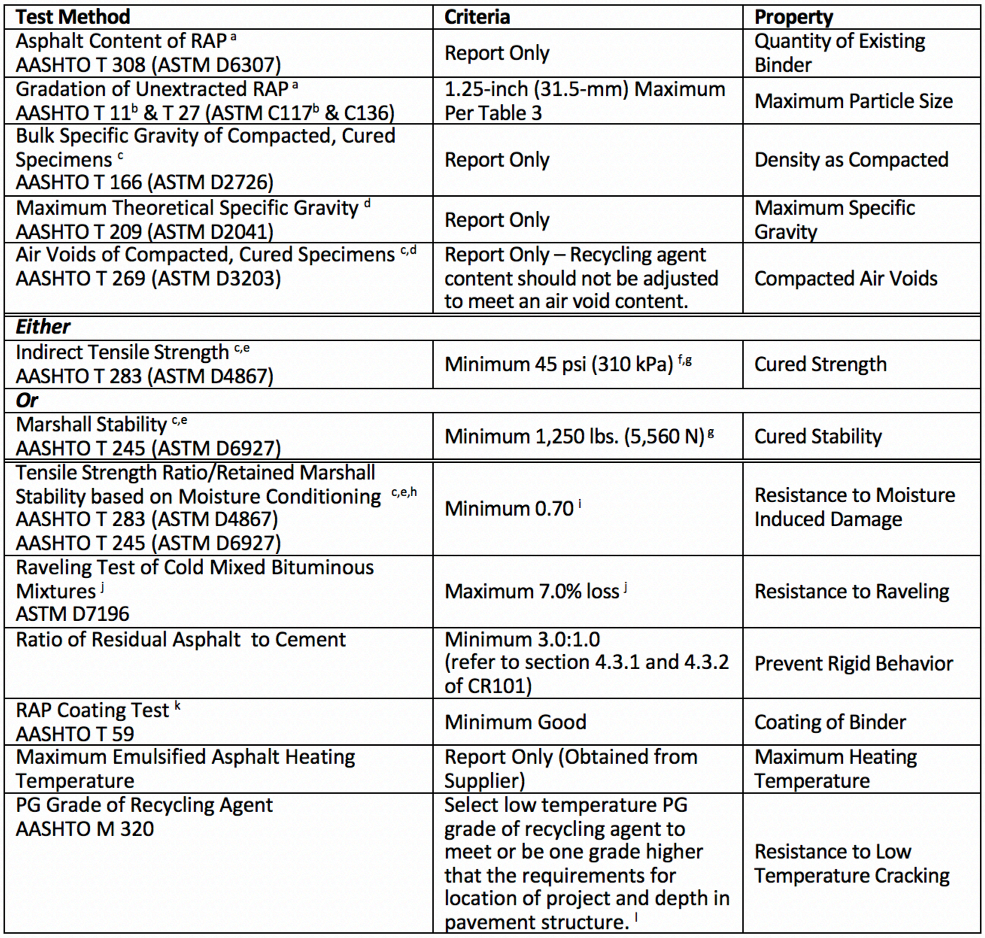
The Cantabro test (AASHTO TP 108) is a common practice for durability evaluation of open graded friction courses (OPFC) and more recently has been used to evaluate conventional asphalt concrete mixtures (Cox et al., 2015). The test presents several advantages including economics and practicality. The procedure consists of determining the mass loss of compacted specimens after 300 revolutions in The Los Angeles drum without the inclusion of steel spheres. Research conducted by Doyle and Howard (2016) determined low variability in the test and the need of a maximum of three replicates for durability characterization.

Table 3 presents an example of the performance criteria define by ARRA CR 201 for the design of cold recycled mixtures stabilized with emulsified asphalt.

2.2.2. Case Studies

Table 4 presents the characteristics of recent field projects where CIR methodologies were employed for the rehabilitation of the pavement structure. Other more relevant case studies with information on performance are presented subsequently.

Table 3. Minimum Cold Recycling Mixture Design Requirements for Emulsified Asphalt (Reprinted from ARRA, 2016)



Test Method	Criteria	Property
Asphalt Content of RAP [a] AASHTO T 308 (ASTM D6307)	Report Only	Quantity of Existing Binder
Gradation of Unextracted RAP [a] AASHTO T 11[b] & T 27 (ASTM C117[b] & C136)	1.25-inch (31.5-mm) Maximum Per Table 3	Maximum Particle Size
Bulk Specific Gravity of Compacted, Cured Specimens [c] AASHTO T 166 (ASTM D2726)	Report Only	Density as Compacted
Maximum Theoretical Specific Gravity [d] AASHTO T 209 (ASTM D2041)	Report Only	Maximum Specific Gravity
Air Voids of Compacted, Cured Specimens [c,d] AASHTO T 269 (ASTM D3203)	Report Only – Recycling agent content should not be adjusted to meet an air void content.	Compacted Air Voids
Either		
Indirect Tensile Strength [c,e] AASHTO T 283 (ASTM D4867)	Minimum 45 psi (310 kPa) [f,g]	Cured Strength
Or		
Marshall Stability [c,e] AASHTO T 245 (ASTM D6927)	Minimum 1,250 lbs. (5,560 N) [g]	Cured Stability
Tensile Strength Ratio/Retained Marshall Stability based on Moisture Conditioning [c,e,h] AASHTO T 283 (ASTM D4867) AASHTO T 245 (ASTM D6927)	Minimum 0.70 [i]	Resistance to Moisture Induced Damage
Raveling Test of Cold Mixed Bituminous Mixtures [j] ASTM D7196	Maximum 7.0% loss [j]	Resistance to Raveling
Ratio of Residual Asphalt to Cement	Minimum 3.0:1.0 (refer to section 4.3.1 and 4.3.2 of CR101)	Prevent Rigid Behavior
RAP Coating Test [k] AASHTO T 59	Minimum Good	Coating of Binder
Maximum Emulsified Asphalt Heating Temperature	Report Only (Obtained from Supplier)	Maximum Heating Temperature
PG Grade of Recycling Agent AASHTO M 320	Select low temperature PG grade of recycling agent to meet or be one grade higher that the requirements for location of project and depth in pavement structure. [l]	Resistance to Low Temperature Cracking

Table 4. Examples of Recent Local and State Cold Recycling Projects

Year	Agency	Location	Project	Length (mi)	Existing Condition	Treatment Depth (in)	Overlay Type & Depth	Additive Type	Est. Cost Savings
2007	Barnes County	Barnes County, ND	Kathryn Rd S	9.5	Rutting, transverse cracking	4	Chip Seal	Emulsion	55%
2007	Oklahoma DOT	Beaver and Harper Counties, OK	US 412	0.3	Transverse cracking	3-4	2-3 in. HMA	Emulsion CSS-1	–
2010	City of Palm Desert	Palm Desert, CA	Residential streets	950k ft^2	Severe cracking	2.5-4	1.25 in. asphalt rubber WMA	Emulsion PASS-R	$450k

Table 4. Continued

Year	Agency	Location	Project	Length (mi)	Existing Condition	Treatment Depth (in)	Overlay Type & Depth	Additive Type	Est. Cost Savings
2010	Illinois DOT	Astoria to Summun, IL	US 24	2.3	Extremely poor condition	2	2 in. HMA	Emulsion	$250k
2010	Texas DOT	Ochiltree County, TX	US 83	6.1	Fatigue and longitudinal cracking	5	1.5 in. HMA	Emulsion CSS-1H	30 - 50%
2011	Los Angeles Department of Public Works	Los Angeles County, CA	Angeles Forest Highway	25	Poor condition	3	1.5 in. asphalt rubber HMA	Emulsion PASS-R	40%
2011	Utah DOT	Bluff, San Juan County, UT	US 191	9	Block cracking	3	Fog seal	Emulsion + Lime	–
2012	Los Angeles Department of Public Works	City of Lancaster, CA	50th St. W btw K Ave. and M-8 Ave.	2	Poor condition	1-2	1.5 in. Asphalt rubber HMA	Emulsion	–
2013	City of Glendale	Glendale, CA	Central Avenue	0.5	Poor condition	4-5	2-3 in. asphalt rubber HMA	Emulsion PASS-R	30 - 35% ($340k)
2013	Delaware River Joint Toll Bridge Commission	Solebury Township, PA, and Delaware Township, NJ	Rte. 202	5	Rutting, alligator cracking	8-6	2 in. HMA	Foamed asphalt	60%
2013	Texas DOT	Hemphill County, TX	US 83	6.1	Rutting, transverse cracking, delamination	4	1.5 in. HMA	Emulsion + Lime Slurry	30 - 50%
2013	West Virginia Division of Highways	Morgantown, WV	Monogalia CR 53/Fort Martin Rd.	1.8	Cracking, potholes, delamination	6	2 in. HMA	Emulsion CSS-1h + Portland cement	–
2014	Lassen and Plumas counties	Sierra Nevada Mountains, CA	Mooney Rd. btw Hwy. 36 and Hwy. 44	7	Rutting	3	20% RAP HMA	Emulsion HFMS-2p	$296k

2.2.2.1. *Iowa CIR Long-Term Performance Evaluation*

In 2007, the Iowa Highway Research Board in collaboration with the Iowa DOT sponsored field and laboratory performance evaluations of 24 CIR rehabilitated roads. Of the total sample, 18 roads were constructed between 1986 and 1998 and initially investigated by Jahren et al. (1998). The remaining six roads were constructed between 1999 and 2004.

Researchers evaluated the influence of various external factors such as traffic level, support condition and age on performance. Roads carrying an annual ADT from 0 to 800 vehicles were classified as low traffic volume and those with more than 800 annual ADT were regarded as high traffic volume. Similarly, researchers created two categories for the support condition according to the subgrade elastic modulus (SEM): adequate for an SEM above 5,000 psi or inadequate for an SEM below 5,000 psi.

In order to properly compare the performance of the pavements with the results previously obtained by Jahren et al. (1998), researchers performed the same series of tests, including collecting qualitative and quantitative surface distress data, defining the support condition based on field deflection and determining the engineering properties of the CIR materials through a series of laboratory tests conducted on field cores.

A pavement distress survey was performed on each road using an Automated Image Collection System (AICS), which allowed for an efficient evaluation of the pavement surface while traveling at highway speed. The dimensions/areas of cracks and other distresses were measured and the Pavement Condition Index (PCI) calculated. A field deflection test was performed using FWD; data was acquired every 100 ft. on a 1,500 ft. long section of the road. Through back-calculation, the elastic modulus of the pavement layers was determined and related to the support condition. All FWD data was analyzed assuming a tree layer pavement structure comprised of a HMA surface layer over a CIR layer and a foundation layer (FND) as shown in Figure 10.

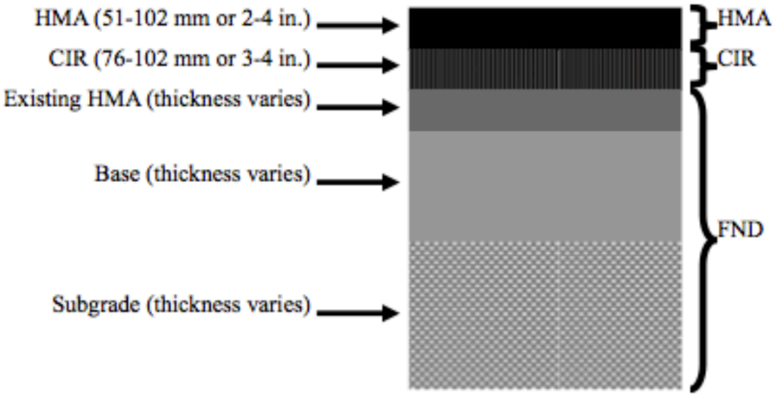


Figure 10. 3-Layer Model Cold in Place Recycling (CIR) Pavement (Reprinted from Chen and Jahren, 2007)

For the laboratory investigation, researchers employed six 4.0 in. (100 mm) diameter field cores that were extracted every 300 ft. including three field cores from the right wheel path and three field cores from the center of the lane between wheels paths. The CIR layers were isolated from the top and bottom layers by trimming, which yielded a 4.0 in. (100 mm) diameter by 2.0 in. (50 mm) tall test specimen. Indirect Tensile Strength (IDT), Air Voids (Va), Complex Shear Modulus (G*), Flexural creep Stiffness (S) and Stiffness-Time relationship (S(t)) were measured on the trimmed CIR field cores.

A statistical analysis was done to correlate field pavement performance (PCI), laboratory test results (IDT, Va, G*, S(t)) and external factors (traffic level, support condition and age). Separate multivariable models were developed for: (a) all roads; (b) high traffic roads and (c) low traffic roads. The results for the first model indicated better pavement performance for higher Va content, lower CIR modulus and lower traffic load. For the second model, the analysis displayed better pavement performance for lower CIR modulus and higher IDT, while for the third model better performance was observed for higher G* and lower CIR modulus. All three models showed better pavement performance with lower moduli and/or higher Va in the CIR layer, suggesting the action of CIR as a stress-relieving layer, a concept previously supported by Abd El Halim (1986). Additionally, the high values of IDT and G* determined on the regressions for the low and high traffic roads models suggest good moisture and rutting resistance of the CIR layer.

2.2.2.2. Virginia I-81

In 2011 the Virginia Department of Transportation (VDOT) completed the rehabilitation of Interstate Highway 81 (I-81) in Augusta County near Staunton, VA (Diefenderfer and Apeagyei, 2014). Three in-place recycling methodologies including FDR, CIR and CCPR were included in the project. The CIR and CCPR methodologies used 1.0% hydraulic cement and 2.0% foamed asphalt, whereas 3.0% of a mixrture of lime kiln dust and hydraulic cement were employed in the FDR process. VDOT was interested in gaining experience with these types of rehabilitation methodologies with regard to mixture design, QA procedures and field evaluation. In addition, the performance of the section was monitored during the first three years after construction via ride quality and rutting measurements with good observed performance despite high traffic volumes on this four- lane divided section of I-81.

Ground penetrating radar and FWD measures were also conducted to verify the thickness and structural soundness of the pavement after construction. From these evaluations, the structural coefficient for the CIR was 0.39, for CCPR between 0.37-0.44 and the combined structural layer coefficient for CCPR and FDR 0.37. The typical structural layer coefficients recommended by AASHTO for these types of materials are between 0.25-0.35 with FDR tending to be on the lower end and CIR/CCPR on the higher end of the range (AASHTO 1993).

The laboratory tests conducted on the materials collected prior, during and after construction (field cores obtained 3 and 20 months after construction) included gradation, resilient modulus, indirect tensile (IDT) strength, dynamic modulus and flow number. A mixture design procedure to determine the optimum moisture content, density at optimum moisture content and the selection of the recycling agent content was done for all mixtures. The CIR and CCPR mixtures were designed in accordance with the Wirtgen manual (Wirtgen 2006). Several foaming water contents were used in a laboratory-scaled foamer to determine the optimum water content for the PG 64-22 binder. Trial mixtures were prepared by compacting in the SGC using a 4 in. (100 mm) diameter mold to a predetermined density equivalent to the density that would be obtained with 75 blows in the Marshall compactor. The 2.5 in. (62.5 mm) tall specimens were cured in an oven at 40°C for 72 hours before IDT strength testing. The specimens with 1.0% hydraulic cement and 2.0% foamed asphalt achieved the minimum IDT strength of 45 psi. For the FDR materials the optimum hydraulic cement plus lime kiln dust content (i.e., 3.0%) was determined via maximum unconfined compressive strength of 300 psi to control cracking.

The right lane of the section was treated with FDR plus CCPR and an asphalt overlay, whereas the left lane was constructed with CIR and an asphalt overlay. No tack coat was applied between layers. During construction, QA and acceptance testing of the CIR and CCPR mixtures included depth of the recycled layer, gradation, recycling agent dose, dry and wet IDT strength and compacted density. The requirement for dry IDT strength was 95% of the design value (i.e., 48.5 psi) and the TSR was only reported. Additional laboratory testing using materials collected during construction and field cores was also performed. The cores were used to determine gradation, binder content, density, IDT strength, resilient modulus and flow number. The results from the laboratory evaluation indicated similar performance between CCPR and CIR specimens.

Field evaluation of rut depth and ride quality showed minimal rutting (<0.01 in. [2.5 mm]) after 34 months in-service. In addition, the ride quality improved from the time of construction (i.e., IRI = 72 in./mile [114 cm/km]) to after about 34 months after construction (i.e., IRI = 45-56 in./mile [71-89 cm/km]). The CCPR over FDR had lower IRI values than the CIR; however, VDOT could not conclude that the differences in IRI values were exclusively due to the different treatments, since the structure of the pavement was slightly different in terms of thickness of the layers. The structural capacity of the layer seemed to also improve with time as demonstrated by larger backcalculated structural numbers from FWD measurements. VDOT will continue to monitor the long-term performance of this section of I-81.

2.2.2.3. VDOT Test Sections at the NCAT Test Track

In 2012, VDOT tested three pavement structures (N3, N4, S12) at NCAT test track in order to evaluate the performance of CCPR and FDR recycling technologies under heavy traffic loading conditions (10 million 18 kip equivalent single axle loads – ESALs). The test sections were 200 ft. long and were comprised of a 5.0 in. (127 mm) CCPR base under virgin asphalt concrete (AC) overlays 4.0 (100 mm) or 6.0 in. (150 mm) thick. A cement stabilized base was included under the CCPR layer in one of the sections to simulate the FDR layer (Figure 11).

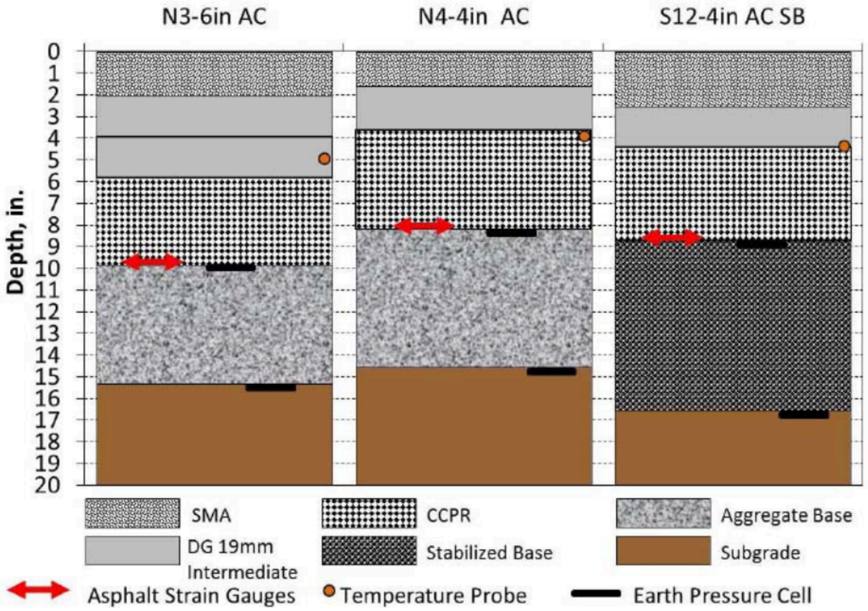


Figure 11. Schematic of the Pavement Structure of the VDOT Section at the NCAT Test Track (Reprinted from Diefenderfer et al., 2016).

By means of gauges, probes and cells embedded during construction within the pavement structure, as shown in Figure 11; strain, temperature and pressure were recorded at various depths. Besides the instrumentation, researchers conducted gradation and binder content tests on loose CCPR material obtained during construction of the test sections. In addition, dynamic modulus tests were conducted on specimens fabricated from the same material using an SGC. To assess field performance, rut depth and ride quality (i.e., IRI index) were measured employing a vehicle mounted censor. Additionally, measurements of structural capacity were made at four locations within each test section using FWD.

The time required to apply the defined traffic load was two years, after which the researchers found no observable surface distresses in any of the test sections. The strain measurements at 68ºF (20ºC) showed an average deformation on Section N3 that was 40 percent lower than that of Section N4; whereas Section S12 displayed an average strain at 68°F (20°C) that was 69 and 49 percent lower than the measurements recorded in Sections N3 and N4, respectively. According to the strain response, researchers ranked the sections from better performance to worst as: S12 > N3 > N4. In addition, a time-increasing strain response for Section N4 was detected, while Section N3 remained constant along the loading period. This difference in strain response was attributed to the presence of damage in Section N4, which was prevented in Section N3 due to the additional thickness of the surface layer. Nevertheless, researchers concluded that all sections were suitable for high traffic applications since less than 0.3 in. (7.6 mm) rutting was measured on them.

3. MATERIALS SELECTION AND CHARACTERIZATION

The materials employed for the production of recycled asphalt mixtures with high contents of RAP (i.e. 60%, 80%, 100%) are shown in Table 5. This chapter describes the raw material selection and characterization.

Table 5. Selected Materials for Mixture Production

Material	Type	Product Description	Material Code	Material Name
Aggregates	Limestone	S1A Stone	C-41	#78 Stone
		Screenings	F-22	W-10 Screenings
	Granite	S1A Stone	C-47	#78 Stone
		Screenings	F-22	W-10 Screenings
RAP	Stockpile 1-09	Limestone	STK 09	
	Stockpile 1-16	Limestone/Granite	STK 16	
Binder	Asphalt binder	PG 52-28	916-52	

To understand the performance of the recycled mixtures, it was important to characterize the individual mixture components: aggregate, binder and RAP by means of standard laboratory tests as described subsequently.

3.1. AGGREGATES

Two aggregate sizes were used for each aggregate type: intermediate size stone (#78) and fine screenings (W-10). The particle size distribution for each of these materials was provided by FDOT and verified employing the *Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates* per AASHTO T-27. The results showed minimal differences between gradations provided by FDOT and the ones obtained in the laboratory. These gradations can be found in Appendix A.

Table 6 presents the Oven dry Bulk Specific Gravity ($G_{S\,(OD)}$) of the aggregates provided by FDOT. Other physical properties of the aggregates are presented in Appendix B.

Table 6. Aggregates Oven Dry Bulk Specific Gravity (GS $_{(OD)}$)

Aggregate type	Product Name	$G_{sb\,(OD)}$ (-)
Limestone	#78 Stone	2.407
	W-10 Screenings	2.520
Granite	#78 Stone	2.775
	W-10 Screenings	2.740

3.2. BINDER

The binder used correspond to a PG 58-22 in accordance to FDOT Standard Specification, Section 334-2.3.5: *Asphalt Binder for Mixes with RAP*, which identifies this binder grade as that required for the production of mixtures with high RAP content.

Following the methodology defined in AASHTO M 320: *Standard Specification for Performance-Graded Asphalt Binder*, measurements of stiffness and phase angle were conducted before and after rolling thin film oven (RTFO) and pressure aging vessel (PAV) in the dynamic shear rheometer (DSR). Binder stiffness and relaxation after RTFO and PAV were also investigated at low temperatures in the bending beam rheometer (BBR). Table 7 displays the determined continuous high and low temperatures PG of the asphalt binder. Appendix D presents detailed results for the binder PG determination.

Table 7. Binder PG 52-28 Continuous Grade

Binder Type	Continuous Grade	
	High-Temp PG (°C)	Low-Temp PG (°C)
Virgin Binder PG 52-28	56.9	-31.3

Using the information obtained from the BBR, the ΔT_c parameter was estimated as 0.8°C for the PG 52-28 binder (Figure 12). This parameter corresponds to the difference between low temperatures where the binder reaches the standard thresholds for stiffness (i.e., S = 300 MPa) and relaxation (m-value = 0.3) and is an indicator of binder quality with regard to its resistance to low temperature cracking.

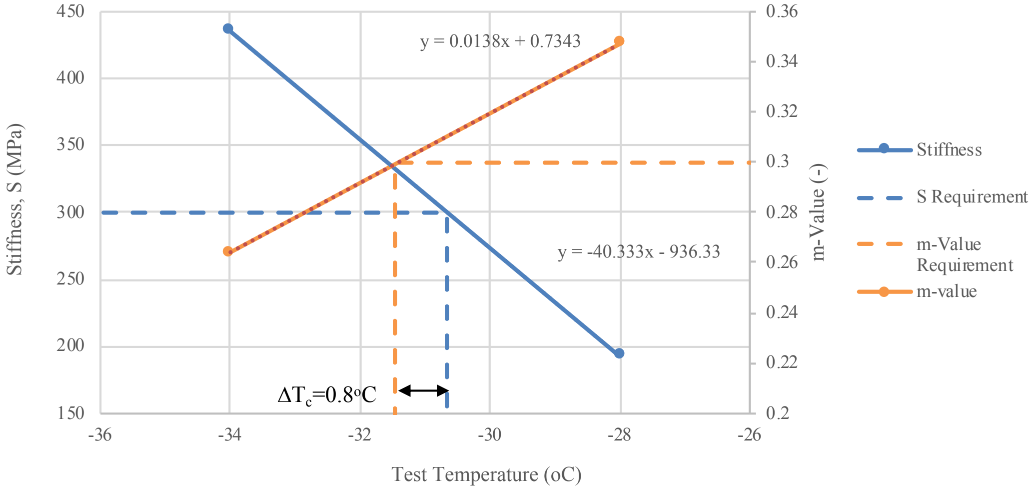


Figure 12. PG 52-28 ΔT_c Parameter Estimation

The asphalt binder aging process was characterized employing the Glover-Rowe (G-R) parameter. Master curves before and after RTFO plus 20, 40 and 60-hour PAV aging were developed in the DSR and then the RHEA™ software was used to estimate complex modulus (G*) and phase angle (δ) at a temperature of 59°F (15°C) and load application frequency of 0.005 rad/s.

The G-R parameter was calculated employing Equation 1 and plotted in the Black space diagram to be compared with the damage thresholds that define the onset and propagation of cracking (Figure 13). The limits are 26 psi (180kPa) for damage onset and 87 psi (600 kPa) for significant damage and correlate to low asphalt ductility values of 2.0 in. (50 mm) and 1.2 in. (30.5 mm), respectively, for field sections located in a PG 58-28 climate (Kandhal 1977, Glover et al. 2005).

The results indicated a quick deterioration of the binder with aging. After short term aging (i.e., RTFO) the binder is right on top of the damage onset curve and, after RTFO plus 40 and 60-hour PAV, the G-R parameters are beyond the significant damage threshold curve, with magnitudes of modulus and phase angle similar to those observed in aged binders extracted and recovered from RAP Stockpile 1-09 and 1-16, respectively, that are also shown in Figure 13.

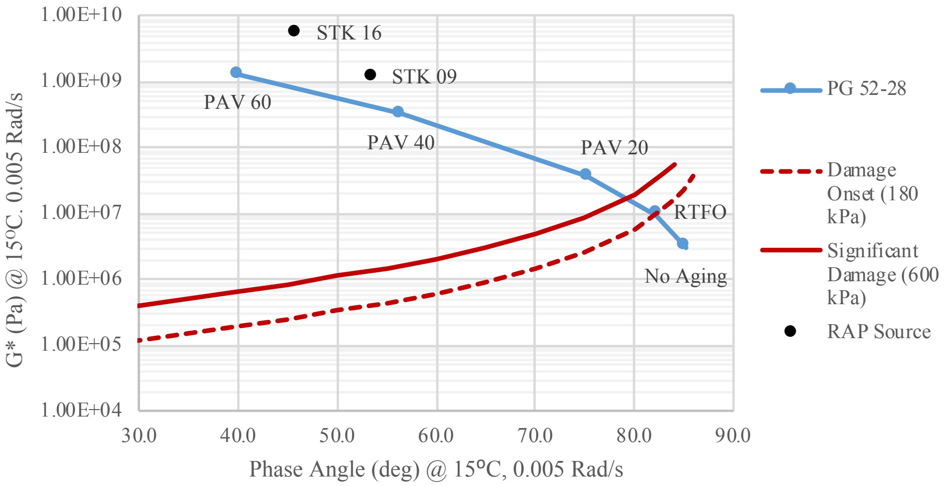


Figure 13. Aging Evaluation of Binder PG 52-28 in Black Space Diagram

3.3. RECYCLED ASPHALT PAVEMENT (RAP)

Two RAP sources, Stockpile 1-09 (limestone) and Stockpile 1-16 (granite/limestone), were used in the fabrication of the recycled mixtures. Binder content and calibration factors were determined for each RAP source following Florida test method FM 5-563: *Quantitative Determination of Asphalt Content from Asphalt Paving Mixtures by the Ignition method.* Table 8 summarizes the results for each RAP source and Appendix E provides the detailed calculations.

Following the methodology defined in AASHTO M 320, measurements of stiffness and phase angle in the DSR and BBR were conducted on samples of binder extracted and recovered from each RAP source. The RAP binder extraction was performed following FDOT test methods FM 5-524: *Reflux Extraction of Bitumen from Bituminous Paving Mixtures* and FM 3-D5404*: Recovery of Asphalt from Solution Using the Rotovapor Apparatus*. Due to pre-existing advanced aging, the characterization of the RAP binders did not include RTFO and PAV tests. Table 8 presents the high and low temperature of the PG determined for each RAP binder.

Table 8. RAP Characteristics

RAP Type	RAP Aggregate Type	Binder Content	RAP Binder Continuous Grade		PG
			High-Temp PG (ºC)	Low-Temp PG (ºC)	
RAP Stockpile 1-09	Limestone	5.4%	96.3	-15.6	94 -10
RAP Stockpile 1-16	Granite / Limestone	4.8%	99.0	-19.2	94 -16

The particle size distribution after binder extraction by means of the ignition oven were provided by FDOT for each RAP source and verified employing the Standard Test Method AASHTO T-27. The results showed minimal differences between the gradations provided by FDOT and the ones obtained in the laboratory. These gradations can be found in Appendix A.

3.4. RECYCLING AGENT SELECTION AND DOSE

This section details the (1) selection, (2) dose estimation and (3) addition method of recycling agents used in the fabrication of the hot recycled mixture specimens. The materials employed in the recycling agent selection task are listed in Table 9. A total of four recycling agents, two classified as organic-based and two classified as petroleum-based, were evaluated.

Table 9. Recycling Agent Types

Category	Description	Product Code
Petroleum-Based	Aromatic Extract	P-1
		P-2
Organic-Based	Bio-based Oil	B-1
		B-2

The four types of recycling agents shown in Table 9 were evaluated to determine the most suitable products to be used in the performance testing of hot recycled high RAP mixtures. One organic-based and one petroleum-based agent were selected so both categories could be assessed and further compared. The selection of the recycling agents was conducted employing blends of virgin binder PG 52-28, extracted and recovered RAP binder and recycling agents. The virgin and RAP aggregates were excluded from the blends in order to only evaluate the interaction of the recycling agents with the binders. A total of eight blends were evaluated as a result of the combination of two RAP binder sources and four recycling agents (Table 8 and Table 9).

The aging susceptibility of the blend was selected as criteria for the recycling agent selection and was quantified by the change after aging in the high-temperature PG (PGH) and Carbonyl Area (CA). The latter is a parameter that quantifies the formation of carbonyl functional groups (C=O bonds) in the binder due to aging.

The blends were subjected to RTFO aging per AASHTO T 240: *Standard Test Method for Effect of Heat and Air on a Moving Film of Asphalt Binder (Rolling Thin-Film Oven Test)*, followed by 40 hours in the PAV per AASHTO R 28: *Standard Test Practice for Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)*.

3.4.1. Initial Recycling Agent Dose

Initial recycling agent doses were estimated using the Equation 2 developed in ongoing NCHRP project 09-58 (Kaseer et al., 2018) and presented previously in the literature review.

Per FDOT Standard Specification, Section 334-2.2: *Superpave Asphalt Binder,* a PG 67-22 binder is required for the production of HMA in the state of Florida ($PGH_{Target} = 67^{o}C$). To estimate the PGH_{blend}, NCHRP 09-58 (Kaseer et al., 2018) was used to develop a blending chart in the form of Equation 3. It requires the determination of the PGH for the virgin binder ($PGH_{V.Binder}$) and RAP source (PGH_{RAP}) employed in the fabrication of the recycled mixture. The recycled binder ratio (RBR) represents the RAP content in terms of replacement of the virgin binder and was computed according to Equation 4.

$$PGH_{Blend} = PGH_{V.Binder} + (PGH_{RAP} - PGH_{V.Binder}) \cdot RBR \qquad \textbf{Equation 3}$$

The binder content of the RAP source (BC_{RAP}) and the optimum binder content (OBC) of the virgin mixture (i.e., with no RAP) were estimated beforehand to compute the RBR of the recycled mixture. The OBC are presented subsequently (Table 21) with the result of the mixture designs.

$$RBR = \frac{\%RAP \ \cdot BC_{RAP}}{100 \cdot OBC} \qquad \textbf{Equation 4}$$

As previously shown, Table 7 and Table 8 contain the $PGH_{V.Binder}$, PGH_{RAP} and BC_{RAP} used to compute the initial dose of recycling agents. A RAP content (%RAP) of 60% was assumed in the calculations. The detailed estimation can be found in Appendix F.

Table 10 presents the resulting PGH_{Blend} and recycling agent doses for each RAP source. These doses were considered as initial estimates and were only used in the selection of the recycling agents. These values were verified in order to guarantee that all blends reached the target PGH.

Table 10. PGH_{Blend} and Recycling Agent Dose Estimate

RAP Source	PGH_{Blend} (°C)	RBR (@ %RAP=60%)	Dose by mass of Total Binder (%)
Stockpile 1-09	75.7	0.47	5.1
Stockpile 1-16	77.1	0.48	5.9

3.4.2. Rheological Characterization

PGH was the rheological parameter used to quantify aging susceptibility of the blends. Following the methodology defined in AASHTO M 320, measurements of stiffness were conducted in the DSR before and after RTFO plus 40–hour PAV. These measurements were performed at increasing temperatures until a minimum blend stiffness of 1.0 kPa was obtained.

Figure 14 and Figure 15 present the change in PGH (i.e., ΔPGH) after RTFO plus 40–hour PAV aging for RAP stockpiles 1-09 and 1-16. Detailed results obtained including the continuous PGH for all blends are shown in Appendix G.

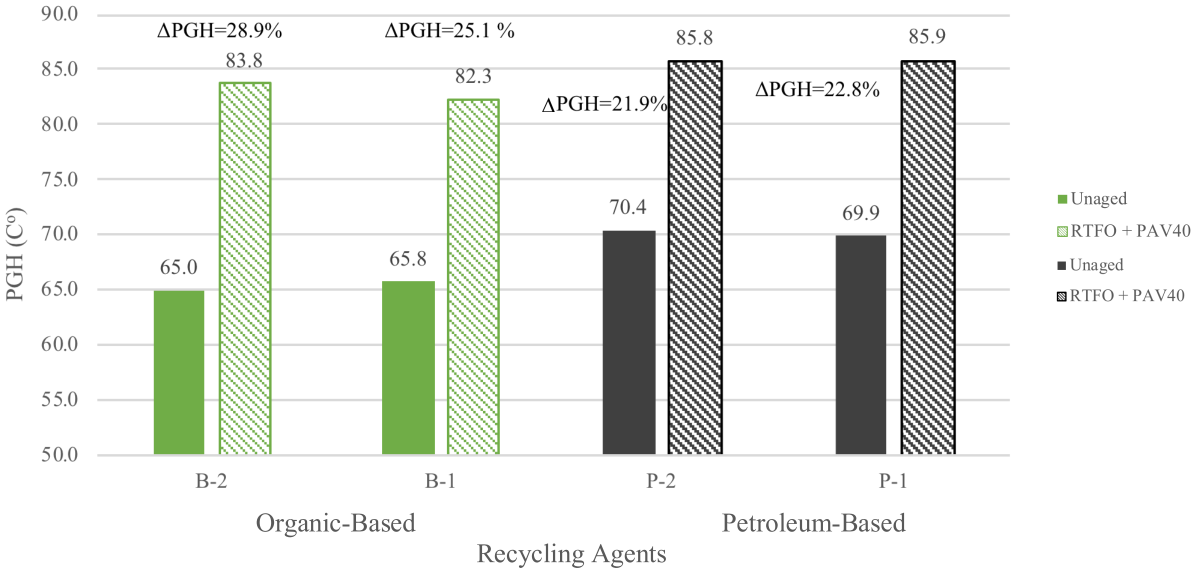


Figure 14. Change in PGH with Aging for Blends with PG 52-28 Virgin Binder + Extracted and Recovered RAP from Stockpile 1-09

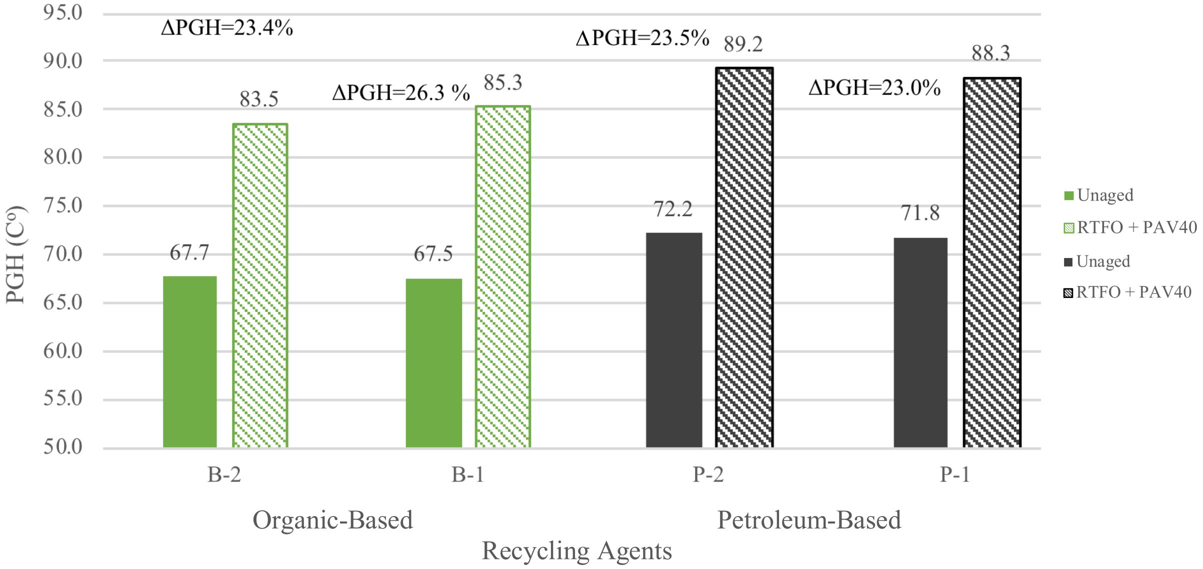


Figure 15. Change in PGH with Aging for Blends with PG 52-28 Virgin Binder + Extracted and Recovered RAP from Stockpile 1-16

According to the information presented in Figure 14 and Figure 15, the blends prepared with extracted and recovered RAP from both sources presented changes in PGH after aging between 23.4 to 28.9% for organic-based recycling agents B-2 and B-1 and between 21.9 to 23.5% for petroleum-based recycling agents P-2 and P-1. This is indicative of the contribution of the recycling agents to improve the performance of the blends after aging. In addition, the results seem to demonstrate that petroleum-based recycling agents are equally or more effective than organic-based recycled agents at minimizing the effect of aging, since a smaller change in PGH was observed in the former. Yet, it is useful to notice that the PGH of the blends prepared with petroleum-based products are about 2–6°C higher than their organic-based counterparts.

The influence of the recycling agents in the change in PGH of the blends after aging differed for each RAP source. Within a recycling agent category, a particular agent generated the largest change in PGH for one RAP source and the lowest change for the other source. This was the case for both recycling agent categories, which led to inconclusive results and not a definite way to select the most effective recycling agent.

3.4.3. Chemical Characterization

Considering the inconclusive rheology results, the evaluation of the blends was approached employing CA, which is a parameter that quantifies the formation of carbonyl functional groups (C=O bonds) in the binder due to aging. This parameter was determined employing Fourier Transform Infrared Sprectroscopy (FT-IR), a method proven effective at evaluating the molecular structure of binders and their change with oxidation. The procedure is based on the premise that different types of chemical bonds absorb light with dissimilar infrared intensity and absorption behavior (Yin, et al. 2017). The CA is defined as the area, in arbitrary units, under the frequency-absorbance curve within the frequency band from 1,820 to 1,650 cm-1. Asphalt binders with greater CA growth after aging are expected to be more susceptible to aging as compared to those with lower CA growth. The CA values were estimated using the equations proposed by Jemison H. et al (1992).

Figure 16 and Figure 17 present the change in CA after RTFO plus 40–hour PAV aging for blends prepared with extracted and recovered RAP from stockpiles 1-09 and 1-16. Detailed results for the change in CA estimation are shown in Appendix G. According to the information presented in Figure 16 and Figure 17, B-1 and P-1 presented a lower change in CA within the organic- based and petroleum-based recycling agent categories, respectively, for both RAP sources. One should note that the changes in CA were approximately the same for the petroleum-based recycling agent products.

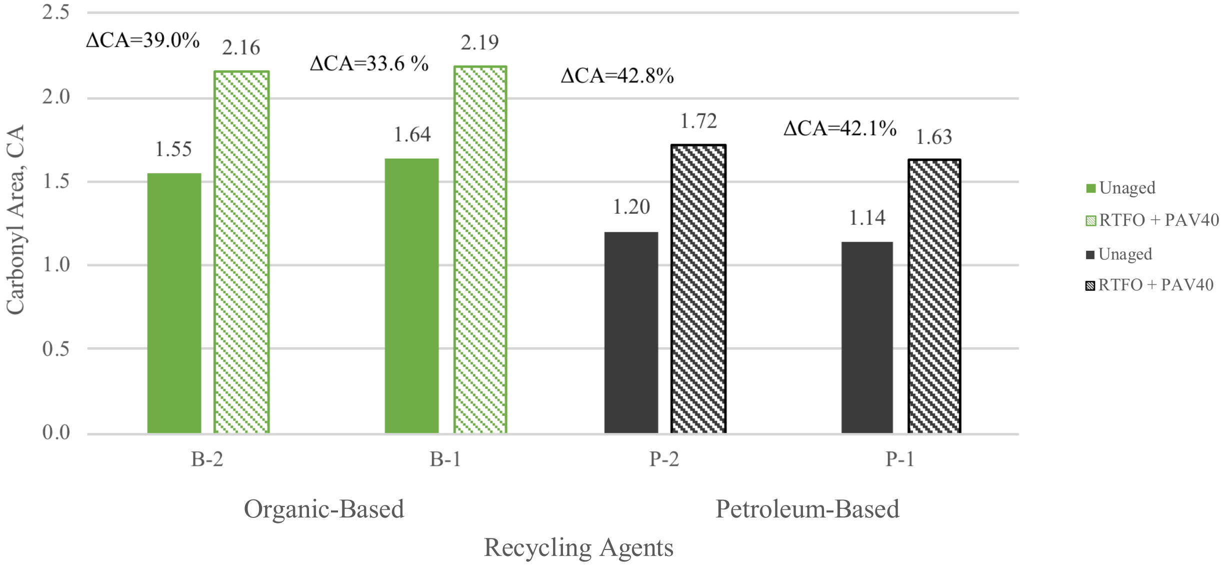


Figure 16. Change in Carbonyl Area Change with Aging for Blends with PG 52-28 Virgin Binder + Extracted and Recovered RAP from Stockpile 1-09

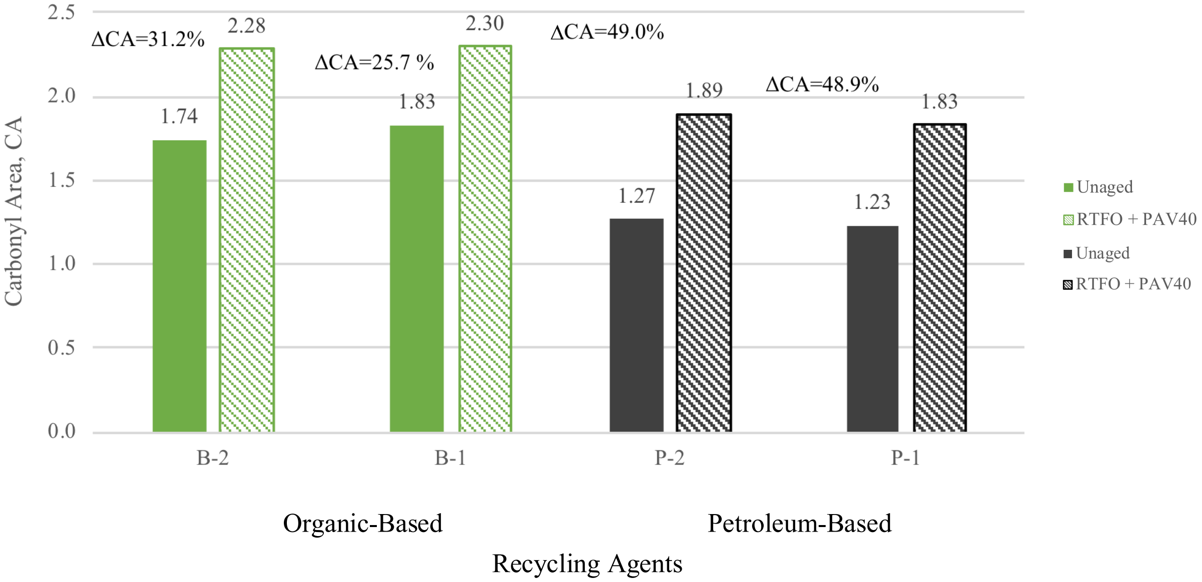


Figure 17. Change in Carbonyl Area with Aging for Blends with PG 52-28 Virgin Binder + Extracted and Recovered RAP from Stockpile 1-16

Considering the information gathered from the rheological and chemical characterization, B-1 and P-1 were selected for further preparation and evaluation of the hot recycled high RAP mixtures. These recycling agents were selected taking into account their lower CA changes within their recycling agent category for each RAP source. In addition, even though the petroleum- based recycling agents presented similar change in CA, FDOT is familiar and has used P-1 successfully in the past, which was a decisive factor in the selection.

3.4.4. Recycling Agent Dose Verification

The initial recycling agent dose estimated using Equation 2 was verified by preparing blends of virgin binder, extracted and recovered RAP binder and 0, 2 and 8% recycling agents, and measuring the unaged and RTFO-aged PGH in the DSR. The validation procedure aimed to match the PGH of the blend to the PGH of the target binder; this is done to avoid rutting problems but provide sufficient cracking resistance (Arambula et al., 2018). As mentioned before, a PG 67-22 is used in Florida to meet climate and traffic demands; thus $PGH_{Target} = 67°C$.

Figure 18 through Figure 21 present the results of the dose verification. The recycling agent doses are reported in percent by mass of total weight of binder and represent the amount of virgin binder replaced by the recycling agent. A total of four blends resulting from the combination of two recycling agents (B-1 and P-1) and two RAP binder sources (Stockpiles 1-09 and 1-16) were evaluated. For the case of the blend with P-1 and extracted and recovered RAP binder from Stockpile 1-16 (Figure 21), an additional recycling agent dose of 14% was included to avoid extrapolating the test data to achieve $PGH_{Target} = 67°C$. Detailed results obtained in the dose verification tests are shown in Appendix H.

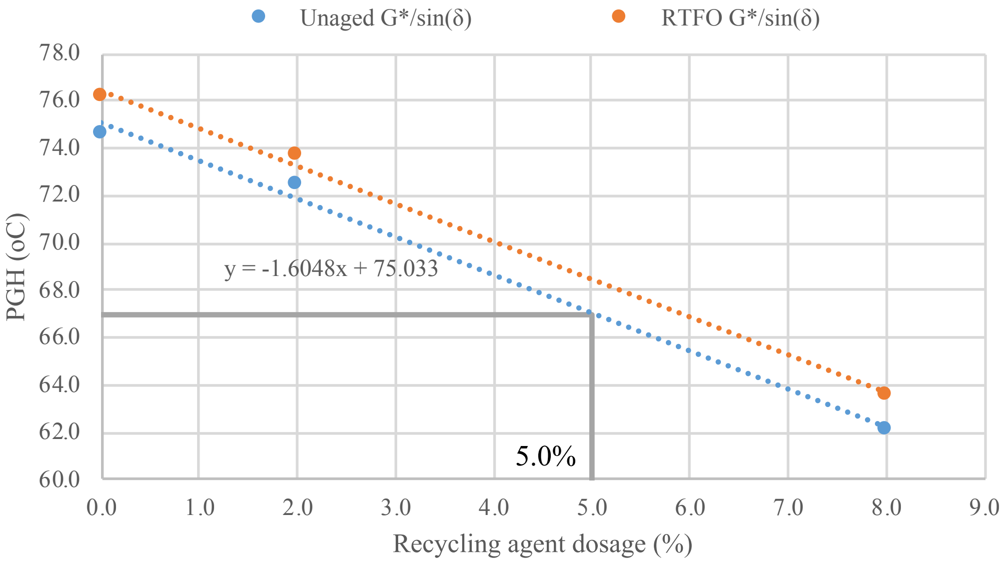


Figure 18. Recycling Agent Dose Verification for Blends with PG 52-28 Virgin Binder + B-1 + Extracted and Recovered RAP from Stockpile 1-09

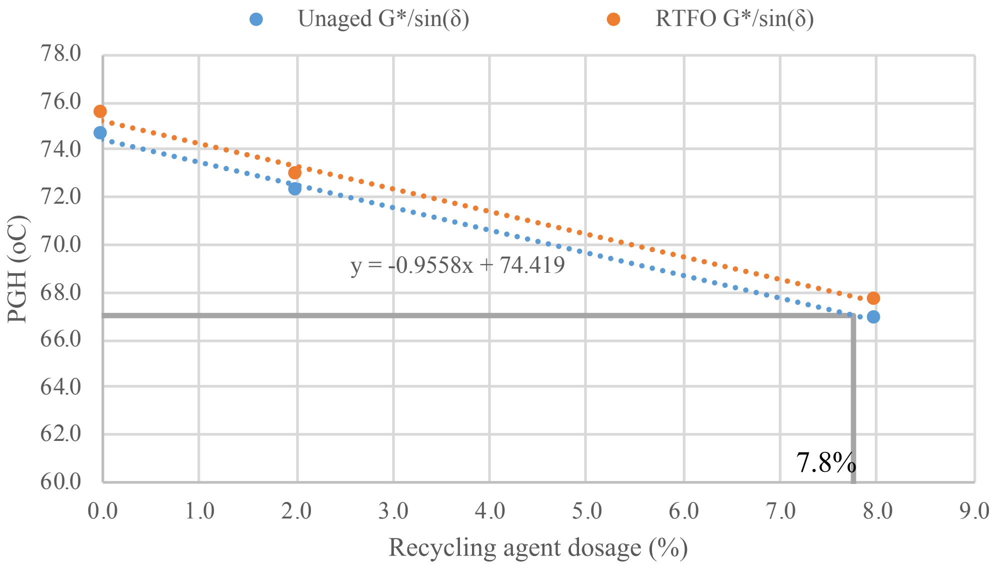


Figure 19. Recycling Agent Dose Verification for Blends with PG 52-28 Virgin Binder + P-1 + Extracted and Recovered RAP from Stockpile 1-09

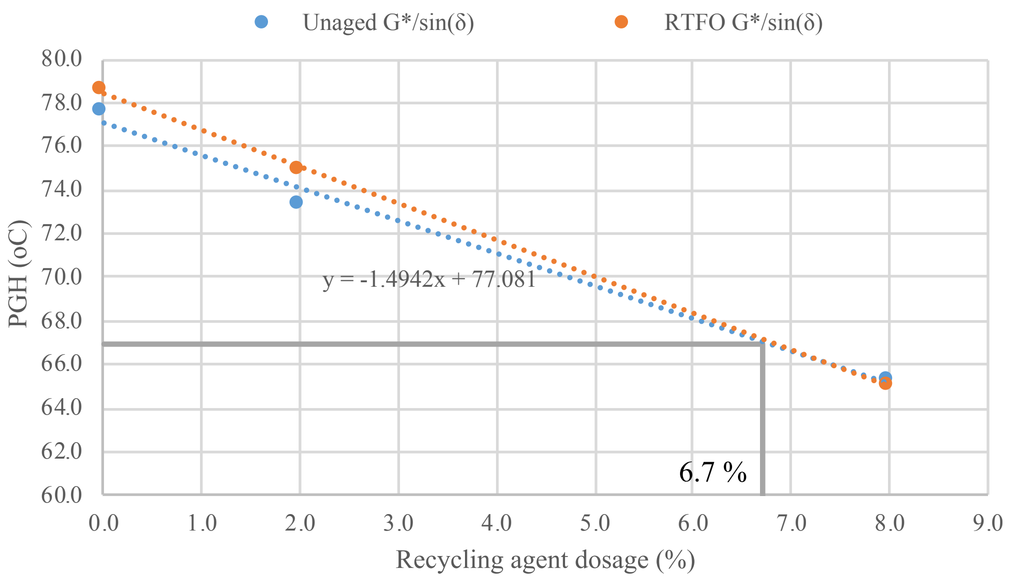


Figure 20. Recycling Agent Dose Verification for Blends with PG 52-28 Virgin Binder + B-1 + Extracted and Recovered RAP from Stockpile 1-16

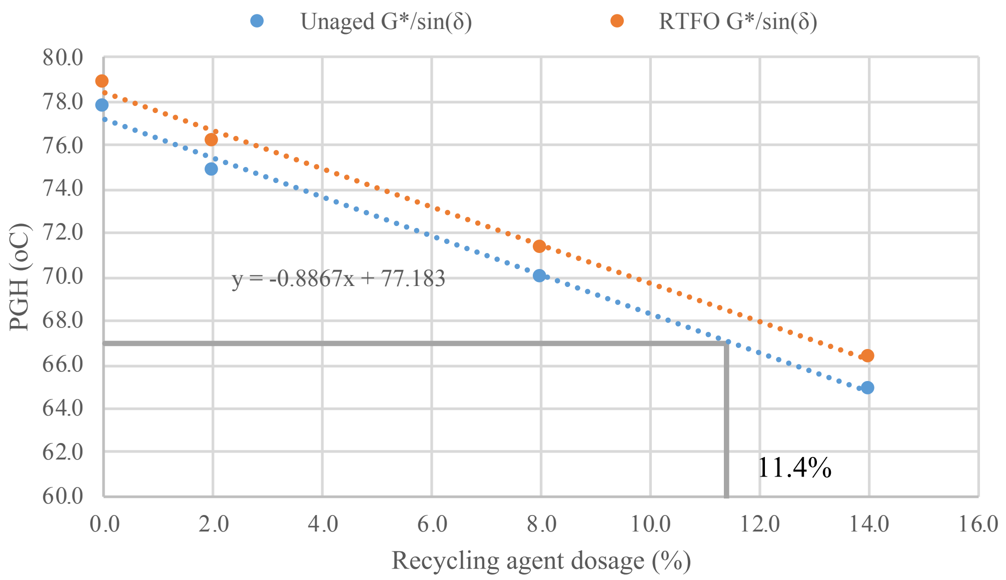


Figure 21. Recycling Agent Dose Verification for Blends with PG 52-28 Virgin Binder + P-1 + Extracted and Recovered RAP from Stockpile 1-16

Table 11 presents the recycling agent doses obtained through the verification process for each blend. The results from the initial recycling agent dose estimates were close for B-1, but not for P-1. The resulting values listed in Table 11 were employed in the fabrication of specimens tested as part of the performance evaluation of hot recycled mixtures.

Table 11. Recycling Agent Dose for Hot Recycled High RAP Mixtures Evaluation

RAP Source	Recycling Agent Type	Dose by mass of Total Binder (%)
Stockpile 1-09	B-1	5.0
	P-1	7.8
Stockpile 1-16	B-1	6.7
	P-1	11.4

3.4.5. Recycling Agent Addition Method

To determine the best method to add the recycling agent to the HMA, workability and coatability tests were conducted on specimens produced with the selected recycling agents (B-1 and P-1) and the RAP from Stockpile 1-16.

The assessment considered two addition methods: (1) adding the recycling agent at the selected dose to the virgin binder (VB + recycling agent, as it is traditionally done in the laboratory and most asphalt production plants) and (2) adding the recycling agent at the selected dose directly to the RAP and letting it "marinate" (RAP + recycling agent). A total of four mixtures resulting from the combination of two recycling agents (B-1 and P-1) and two addition methods were evaluated.

3.4.5.1. Workability

Workability is a property of mixtures that describes how easy it is to place and compact them. The method employed to evaluate this property is based on work done by De Sombre et al. (1998) and employed in project NCHRP 09-53 (Newcomb et al. 2015), which requires measurement of the shear stress and sample height at each gyration during specimen compaction in the SGC.

Specimens 6 in. (152.4 mm) diameter by approximately 4 in. (101.6 mm) height were fabricated employing the high RAP mixture design H-60G-G presented subsequently in the Mixture Design section.

For the RAP + recycling agent addition method, a "marination" period of two minutes was used to let the recycling agent react with the heated RAP, after which both materials were put back into the oven at the mixing temperature (275°F [135°C]) for no more than 8 minutes to allow the RAP to recover lost heat. The pre-heated virgin aggregates and virgin binder at mixing temperature were then added to the marinated RAP to produce the mixture.

The maximum specific gravity of the HMA (G_{mm}) was estimated following Florida's standard test method FM 1-T209: *Maximum Specific Gravity of Asphalt Paving Mixtures*. The results are listed in Appendix I.

The criteria used to evaluate workability included the maximum shear stress (τ_{max}), gyration number to τ_{max} and density energy. All parameters were estimated from the data collected in the SGC. The density energy is a parameter of arbitrary units understood as the required energy for compaction and calculated as the area under the $\%G_{mm}$-N curve from the initial (N_{ini}) to the maximum (N_{Max}) number of gyrations. In this project, $N_{ini} = 6$, according to the standard method

AASHTO M323: *Superpave volumetric mix design guidelines for low volume roads*, and N_{Max} = 300. HMA with higher energy densities are expected to exhibit lower workability when compared to those with lower energy densities. Table 12 and Table 13 present the values determined for the B-1 and P-1 recycling agents, respectively.

Table 12. B-1 Workability Test Results

Addition Method	**Max Shear, τ_{max} (kPa)**	**No. of Gyrations to τ_{max}**	**Density Energy (-)**
VB + recycling agent	413.0	13.0	28,059.7
RAP + recycling agent	408.0	13.0	28,044.8
Relative Difference (%)	-1.2	-	-0.05

Table 13. P-1 Workability Test Results

Addition Method	**Max Shear, τ_{max} (kPa)**	**No. of Gyrations to τ_{max}**	**Density Energy (-)**
VB + recycling agent	383.0	9.0	28,086.9
RAP + recycling agent	396.0	16.0	28,043.8
Relative Difference (%)	3.4	77.8	-0.15

These results show that the recycling agent addition method had no impact on the mixture workability since very small to negligible changes were observed in the selected parameters for both recycling agents. P-1 presented a high relative change of 77.8% in the number of gyrations to the maximum shear stress. However, considering that the net measured difference between addition methods is only 7 gyrations, the two values can be considered practically equivalent, especially when compared to the 300 gyrations employed for specimen preparation.

3.4.5.2. Coatability

The coatability test procedure is based on the premise that aggregates completely coated with asphalt binder will exhibit zero water absorption when submerged for a short period (i.e., 1 hour) since the asphalt film covering the aggregate particles will not allow water permeation. On the contrary, partially coated aggregates are expected to absorb water when submerged, and hence, exhibit a lower coatability index (CI).

Coatability tests were conducted following a protocol proposed by Newcomb et al. (2015) in NCHRP report 807: *Properties of Foamed Asphalt for Warm Mix Asphalt Applications*, which corresponds to a modified procedure of the work originally developed by Velasquez et al. (2010). The protocol for the coatability test can be found in NCHRP report 807 (Newcomb et al. 2015).

According to the test procedure, 8.8 lb. (4000 g.) from the coarse portion of the Granite + RAP mixture were employed as sample mass to conduct the test. The mixtures were produced following the procedure described and employed for the workability tests. However, compaction was not conducted, since the test requires loose mixture specimens. Table 14 present the CI values estimated for each recycling agent and addition method. Detailed calculations of CI are shown in Appendix I.

Table 14. Coatability Test Results

Recycling Agent Addition Method	**C.I.**	
	B-1	**P-1**
Virgin Binder + Recycling Agent	93.1	90.6
RAP + Recycling Agent	84.0	75.6
% Change	-9.9%	-16.5%

The information presented in Table 14 show that the recycling agent addition method did have an impact on the mixture coatability since reductions of the CI were observed for both B-1 and P-1 when the recycling agent was added to the RAP.

Considering the information gathered from the workability and coatability tests, it was decided to follow the more traditional procedure of adding the recycling agent to the virgin binder for further preparation and evaluation of the hot recycled mixtures. This addition method of the recycling agent was selected taking into account that no negative effect on workability was observed and that better CI were obtained.

4. LABORATORY EXPERIMENTS AND RESULTS

The following sections present the results of the laboratory tests conducted for the production and performance evaluation of the recycled mixtures.

4.1. HOT RECYCLED MIXTURES

4.1.1. Mixture Design

The hot recycled mixtures were designed employing the Superpave methodology outlined in FDOT Standard Specification, Section 334: *Superpave Asphalt Concrete* and AASHTO M 323. A virgin mixture for each type of aggregate was designed first in order to find the aggregate gradation and OBC that satisfied all volumetric property requirements. After the virgin mixture design was established, the RAP material was introduced and adjustments to the aggregate gradation and amount of virgin binder were made by taking into account the gradation of the RAP, its binder content and the RAP content in the mixture.

4.1.1.1. *Virgin Mixture Designs*

Aggregate Gradations

For each aggregate type (i.e., limestone and granite), the #78 intermediate size stone and W-10 fine screenings were blended to meet the aggregate gradation requirements established in FDOT Standard Specification, Section 334-1.3: *Gradation Classification*. Three asphalt mixtures are defined based on AASHTO M 323: mixtures with nominal maximum aggregate sizes (NMAS) of 3/8 in. (SP-9.5), ½ in. (SP-12.5) and ¾ in. (SP-19.0).

The combined aggregate blend proportions of the #78 stone and W-10 screenings that met the gradation requirements are shown in Table 15. Figure 22 and Figure 23 show these gradations for limestone and granite blends, respectively. It is noteworthy that the limestone aggregate blend had a NMAS of ¾ in. (19.0 mm), whereas the granite aggregate blend had a NMAS of ½ in. (12.5 mm). Therefore, the mixture produced and requirements verified for the limestone mixture corresponded to SP-19 and for the granite mixture to SP-12.5. Detailed aggregate blend calculations are shown in Appendix C.

Table 15. Aggregate Proportions for the Hot Recycled Virgin Mixture

Aggregate's Blend	**Proportioning**		**NMAS (in), [mm]**	**Superpave Mixture**
	#78 Stone	**W-10 Screenings**		
Limestone	50%	50%	¾, [19.00]	SP-19.0
Granite	40%	60%	½, [12.50]	SP-12.5

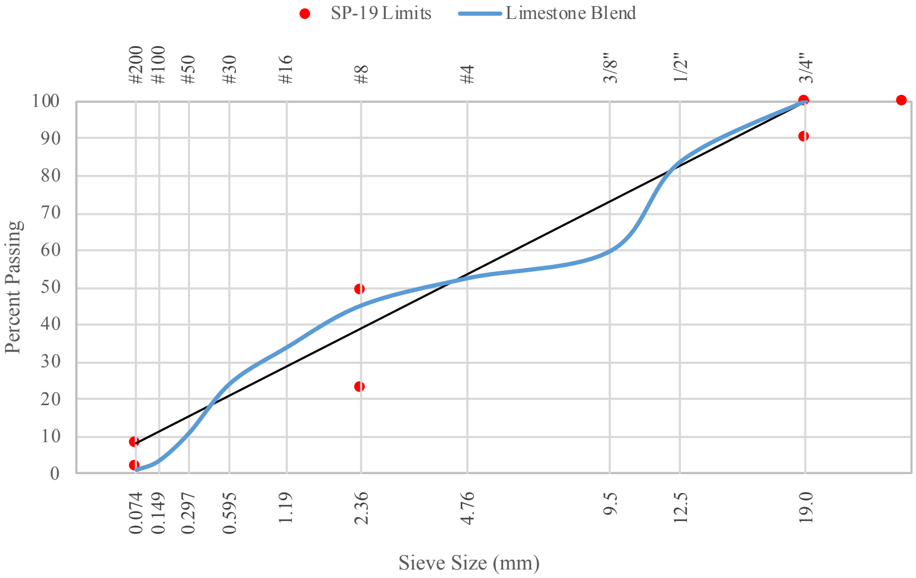


Figure 22. Limestone Aggregate Blend Gradation Curve

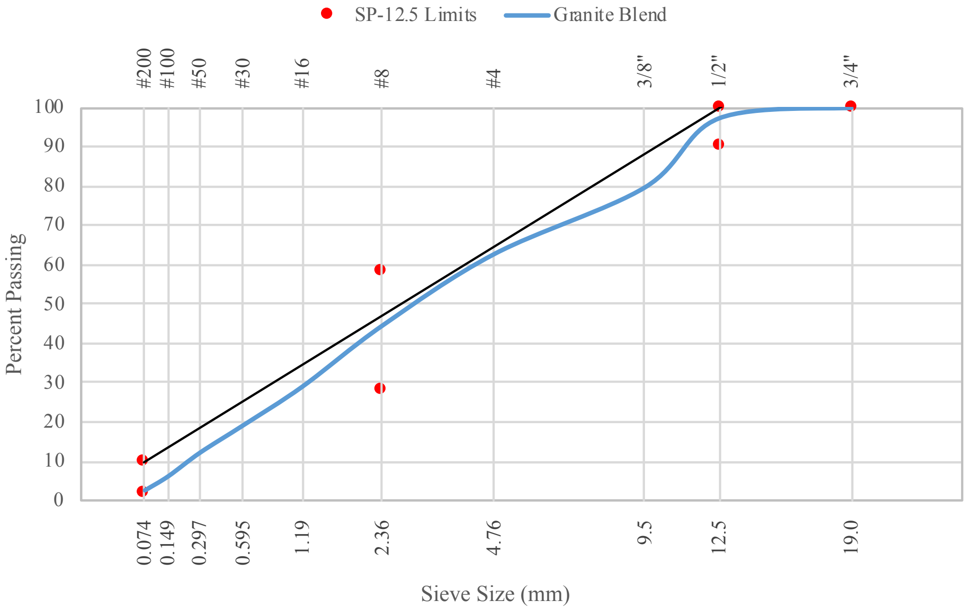


Figure 23. Granite Aggregate Blend Gradation Curve

A washed sieve analysis was performed on each aggregate blend following FDOT standard test method FM 1-T011: *Materials Finer than 75µm (no. 200) Sieve in Mineral Aggregate by Washing*. The material finer than sieve size No. 200 was adjusted for each aggregate blend considering the washed sieve analysis results displayed in Table 16.

Table 16. Washed Sieve Analysis Test Results

Aggregate's blend	Mass change, ΔW (%)	Mass retained at Pan (%)*	Mass < #200 adhered to Larger aggregates (%)
Limestone	1.9	1.1	0.8
Granite	3.5	2.7	0.8

*Mass determined at ordinary sieve analysis AASHTO T-27

Specimen Fabrication for Virgin Mixture Designs

Mixtures with four asphalt binder contents were fabricated for each aggregate blend shown in Table 15, in order to find the OBC that satisfies volumetric requirements specified in AASHTO M323, Table 6, as required by FDOT Standard Specification, Section 334-3.2.5: *Design Criteria*.

Before mixing, the aggregate blends were placed in an oven at 230°F (100°C) and left overnight. A mechanical mixer was then employed to combine the materials at 275°F (135°C) until uniform aggregate coating was observed or a maximum of two minutes of mixing was reached.

Once mixing was complete, the loose mixture was conditioned in the oven for two hours at 275°F (135°C) to simulate the plant production process. After this period, two specimens 6 in. (150 mm) diameter by 4.5 in. (115 mm) height were compacted for each asphalt content in the SGC to N_{Design} = 50 gyrations as established in FDOT Standard Specification, Section 334-3.2.4: *Gyration Compaction*, for a traffic level of less than 0.3×10^6 equivalent single axle loads (ESALs).

After compacting, the specimens were placed on a flat surface and allowed to cool down for at least 24 hours. After this period, the mass of the specimen in air, mass of the specimen soaked in water and the saturated surface dry (SSD) mass of each specimen was determined as required by Florida's standard test method FM 1-T166: *Bulk Specific Gravity of Compacted Asphalt Specimens*. Appendix J presents the bulk specific gravity (G_{sb}), estimated effective specific gravity (G_{se}) and estimated volumetric properties for each aggregate type and asphalt binder content.

Two additional samples at one of the asphalt binder contents were fabricated for each aggregate blend and allowed to cool down at ambient temperature in loose condition. The maximum specific gravity of these mixtures (G_{mm}) was measured following Florida's standard test method FM 1-T209. These results are also listed in Appendix J.

Volumetric Properties of the Virgin Mixtures

Table 17 present the effective specific gravity (G_{se}) values calculated for each mixture type from the measured values of G_{mm}.

Table 17. Virgin Mixtures Effective Specific Gravity (G_{se}), Hot recycling

Virgin Mixture Type	Average G_{se} (-)
Limestone	2.604
Granite	2.818

Figure 24 and Figure 25 show the relationship between air void (AV) content and asphalt binder content of the limestone and granite mixtures, respectively. Plots for the trends of other volumetric properties are presented in Appendix J.

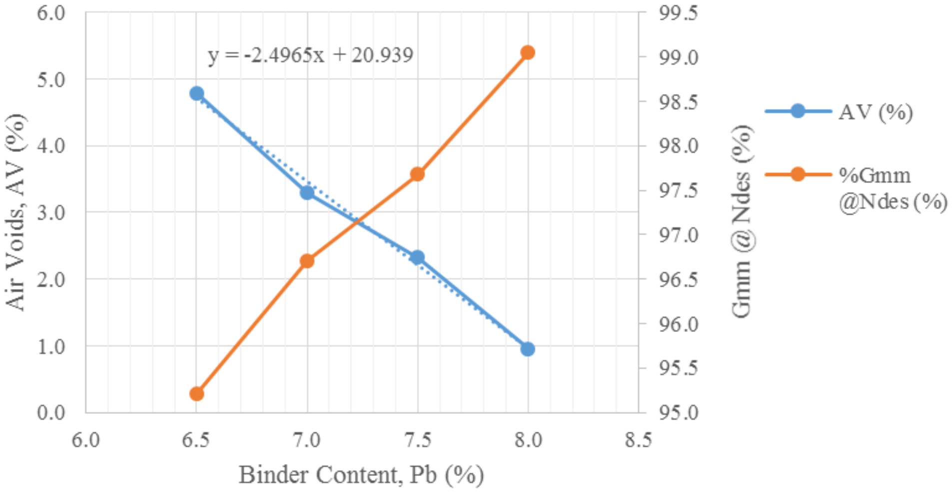


Figure 24. Air Void Content to Asphalt Binder Content for the Limestone Virgin Mixture

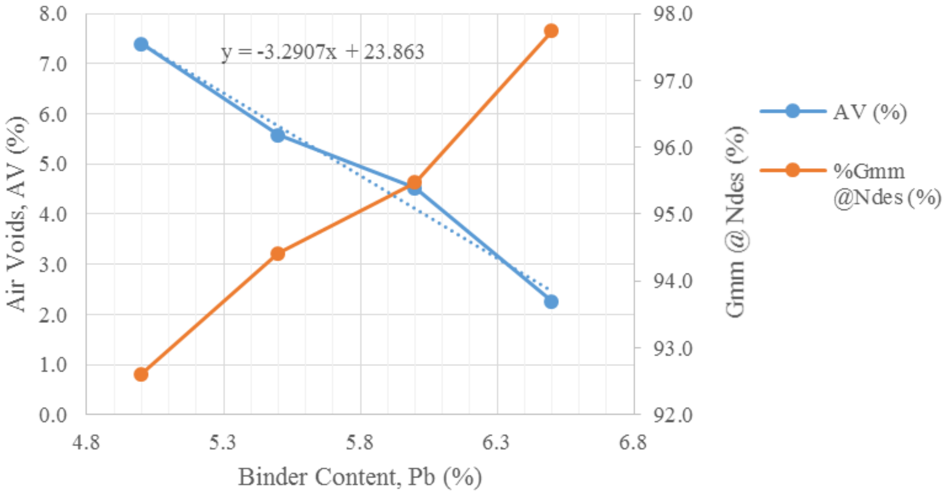


Figure 25. Air Void Content to Asphalt Binder Content for the Granite Virgin Mixture

Table 18 and Table 19 present the OBC for the limestone and granite virgin mixtures determined using the linear equations presented in Figure 24 and Figure 25. According to these relationships, the OBC for the limestone mixture corresponds to 6.8% and for the granite mixture corresponds to 6.0%. Voids in the mineral aggregate (VMA), voids filled with asphalt (VFA) and dust proportion (DP) calculated at the selected OBC are also listed in Table 18 and Table 19. The DP for the limestone mixture is slightly lower (i.e., 0.4) than the limit prescribed in FDOT Standard Specifications (i.e., 0.6 - 1.2). All other volumetric properties are within the specification limits.

Table 18. Limestone Virgin Mixture Volumetric Properties at Optimum Binder Content

Property	FDOT Spec. 334 SP-19	Mixture Design
OBC (%)	-	6.8
AV (%)	-	3.9
VMA (%)	> 13.0	14.6
VFA (%)	70 - 80	73.1
DP (%)	0.6 - 1.2	0.4

Table 19. Granite Virgin Mixture Volumetric Properties at the Optimum Binder Content

Property	FDOT Spec. 334 SP-12.5	Mixture Design
OBC (%)	-	6.0
AV (%)	-	4.4
VMA (%)	> 14.0	17.1
VFA (%)	70 - 80	74.6
DP (%)	0.6 - 1.2	0.7

It is worth mentioning that the volumetric properties (VMA, VFA and DP) at the OBC of 6.0% for the granite mixture (Table 19) are not exactly the same as the ones obtained from the laboratory measurements at 6.0% Pb because the linear equation in Figure 25 was employed to obtain the values listed in Table 19. The final mixture design in FDOT format is presented in Appendix K.

4.1.1.2. Recycled Mixture Designs

The design of the hot recycled asphalt mixtures with high RAP content was done by modifying the virgin mixture design to take into account the after ignition-oven gradation and binder content of the RAP, while maintaining the OBC established for the virgin mixtures (Table 18 and Table 19). As previously mentioned, two RAP sources Stockpile 1-09 (limestone) and Stockpile 1-16 (granite/limestone) were used in combination with the virgin limestone and granite aggregates.

The binder content and calibration factors were determined for each RAP source following Florida test method FM 5-563 as described in the materials characterization chapter (Table 8 and Appendix E).

Three combinations of aggregate type and RAP source, hereafter referred to as aggregate blends, were selected for the design of hot recycled mixtures (Table 20). Therefore, the RAP, #78 stone and W-10 screenings were blended to meet the aggregate gradation requirements established in FDOT Standard Specification*,* Section 334-1.3. The resulting blends proportions are shown in Table 20.

Table 20. Aggregate Blends Proportions for Hot Recycled Mixtures

Aggregate blend	RAP			Virgin Aggregate		
	Source	Aggregate Type	Amount (%)	Type	#78 Stone (%)	W-10 Screenings (%)
ABH-60L-L	Stockpile 1-09	Limestone	60	Limestone	35	5
ABH-60G-G	Stockpile 1-16	Limestone / Granite	60	Granite	20	20
ABH-60L-G	Stockpile 1-09	Limestone	60	Granite	35	5

Figure 26 through Figure 28 show the resulting aggregate gradation curves for the aggregate blends presented in Table 20.

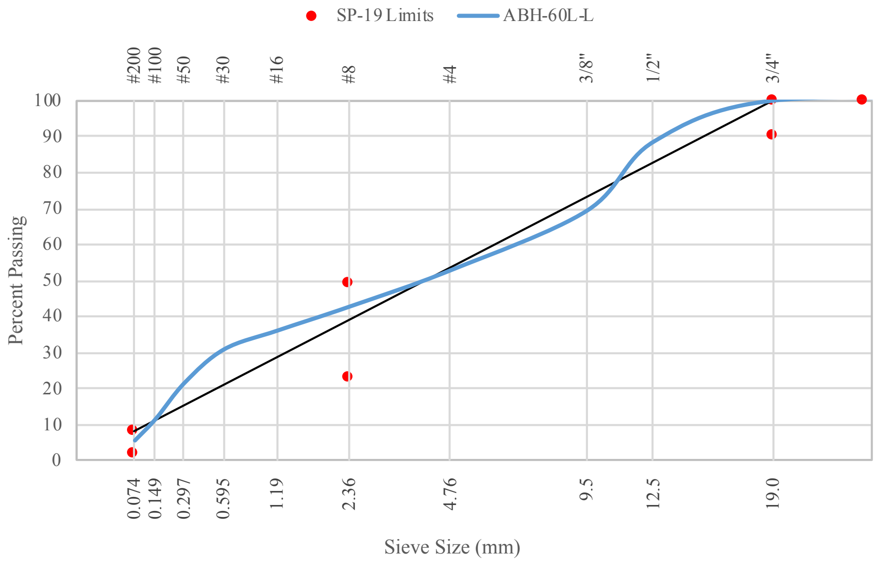


Figure 26. ABH-60L-L Aggregate Blend Gradation Curve

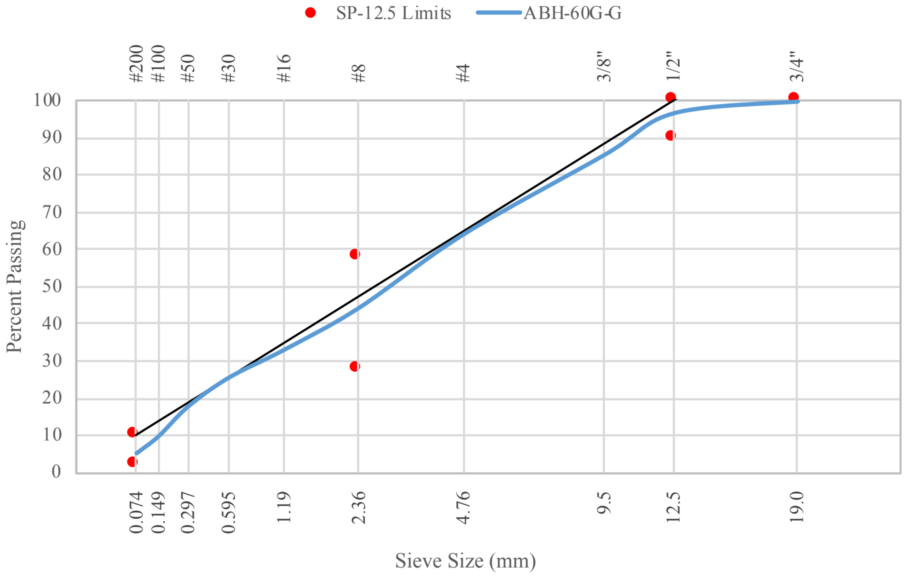


Figure 27. ABH-60G-G Aggregate Blend Gradation Curve

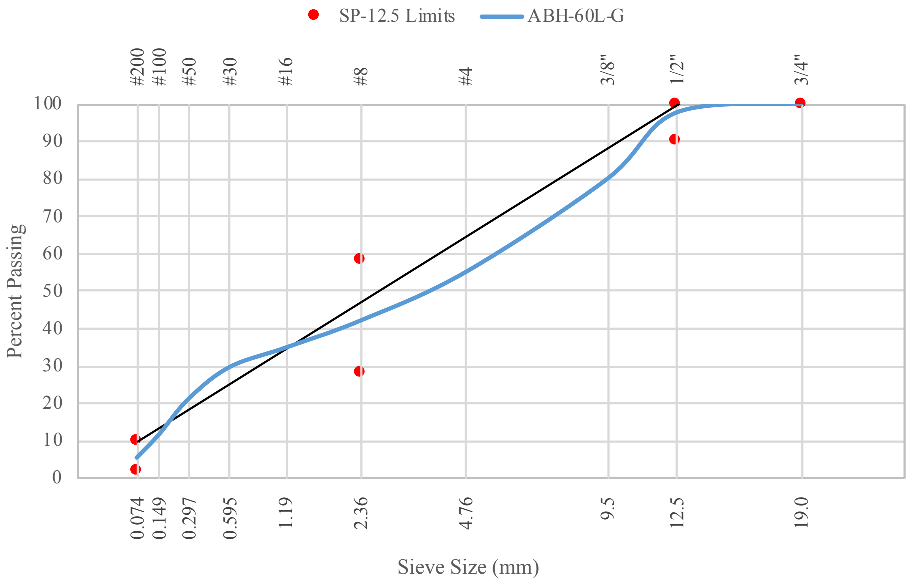


Figure 28. ABH-60L-G Aggregate Blend Gradation Curve

As before, the ABH-60L-L aggregate blend resulted in a NMAS of ¾ in. (19.0 mm), whereas the ABH-60G-G and ABH-60L-G aggregate blends resulted in a NMAS of ½ in. (12.5 mm). Therefore, the mixture produced and requirements verified for the ABH-60L-L aggregate blend corresponded to SP-19 and for the ABH-60G-G and ABH-60L-G aggregate blends to SP-12.5. Detailed virgin aggregate and RAP combined aggregate blend calculations are shown in Appendix C.

Table 21 shows the amount of virgin asphalt (PG 52-28) required for the fabrication of the hot recycled mixtures according to the proportioning of RAP and virgin aggregates previously defined and the OBC determined from the virgin mixture design. These reported amounts were estimated assuming that all the asphalt present in the RAP is activated and contributes to the OBC required in the mixture. The mixture designs in FDOT format are presented in Appendix K.

Table 21. Virgin Binder (PG 52-28) Content for Hot Recycled Mixtures

Hot Recycled Mixture	Aggregate's Blend	OBC (%)	RAP		PG 52-28 Virgin Binder	
			% by weight of Aggregates	Binder Content (%)	% by weight of Aggregates	Binder Content (%)
H-60L-L	ABH-60L-L	6.8%	60.0%	5.3%	40.0%	3.6%
H-60G-G	ABH-60G-G	6.0%	60.0%	4.8%	40.0%	3.1%
H-60L-G	ABH-60L-G	6.0%	60.0%	5.3%	40.0%	2.8%

4.1.2. Specimen Fabrication

Specimens for performance testing of hot recycled asphalt mixtures were fabricated for the recycled mixture presented in Table 22. The proportioning of aggregate and RAP determined to meet FDOT Standard Specification, Section 334-1.3 are presented in Table 22. The OBC employed in the mixtures was defined by pairing the virgin aggregate of the mixture designs presented in

Table 21, with the type of virgin aggregate used in each recycled mixture (Table 22). Therefore, limestone virgin aggregate mixtures were assigned an OBC of 6.8% while granite virgin aggregate mixtures were assigned an OBC of 6.0%.

Table 22. Hot Recycled Mixture Types

Mixture ID	RAP Type and Amount		Virgin Aggregate Type Proportioning				OBC	Recycling Agent	
			Limestone		Granite				
	STK 09	STK 16	#78 Stone	W-10 Screenings	#78 Stone	W-10 Screenings		Product	Dose by Mass of Binder (%)
H-60G-G	-	60.0%	-	-	20.0%	20.0%	6.0%	-	-
H-60G-GO	-	60.0%	-	-	20.0%	20.0%	6.0%	B-1	6.7%
H-60G-GP	-	60.0%	-	-	20.0%	20.0%	6.0%	Hydrolene ®	11.4%
H-60L-L	60.0%	-	35.0%	5.0%	-	-	6.8%	-	-
H-60L-LO	60.0%	-	35.0%	5.0%	-	-	6.8%	B-1	5.0%
H-60L-LP	60.0%	-	35.0%	5.0%	-	-	6.8%	Hydrolene ®	7.8%
H-60L-GO	60.0%	-	-	-	35.0%	5.0%	6.0%	B-1	5.0%
H-60L-GP	60.0%	-	-	-	35.0%	5.0%	6.0%	Hydrolene ®	7.8%

Table 23 presents the list of the tests employed in the performance evaluation of recycled mixtures with foamed asphalt. Specimen characteristics and quantities is also provided. A total of 96 specimens were fabricated.

Table 23. Hot Recycled Specimen Characteristics and Quantities

Mixture Property	Test	Standard	Diameter, In. (mm)	Compaction Criteria	Number of Samples per Mixture Type	Total Number of Samples
Moisture Susceptibility	Modified Lottman, Indirect Tension Test (IDT)	FM 1-T283	6 (152.4)	Height: 1.5 in. (38.1 mm)	6	48
Rutting & Moisture Susceptibility	Hamburg Wheel Track Test (HWTT)	AASHTO T 324	6 (152.4)	Height: 2.5 in. (63.5 mm)	4	32
Intermediate Temperature Cracking	Semi-Circular Bending Beam (SCB)	AASHTO TP 124	6 (152.4)	Height: 2.0 in. (50.8 mm)	2	16
Stiffness	Resilient Modulus (M_R)	ASTM D7369	6 (152.4)	Height: 2.0 in. (50.8 mm)	1*	8*

* The M_R test was conducted on specimens fabricated for intermediate temperature cracking evaluation. One additional specimen was fabricated to conduct three replicates of MR test.

Before mixing, the blends of virgin aggregate (#78 stone and W-10 screenings) were placed in an oven at 230°F (100°C) and left overnight. Two hours before mixing, RAP, asphalt (PG 52-28) and aggregate blends were placed together in an oven at the mixing temperature of 275°F (135°C). A mechanical mixer was then employed to combine the materials until uniform aggregate coating was observed or a maximum of two minutes of mixing was reached.

Once mixing was complete, the loose mixture was conditioned in the oven for two hours at 275°F (135°C) to simulate the plant production process. After this period, the set of specimens defined in Table 23 was compacted in the SGC to the compaction criteria specified for each specimen type in the same table (Table 23).

After compacting, the specimens were placed on a flat surface and allowed to cool down for at least 24 hours. After this period, the mass of the specimen in air, mass of the specimen soaked in water and the saturated surface dry (SSD) mass of each specimen was determined as required by Florida's standard test method FM 1-T166. Appendix N presents the bulk specific gravity (G_{sb}) and estimated AV content for each specimen and mixture defined in Table 22 and Table 23.

Two additional samples were fabricated for each hot recycled mixture displayed in Table 21 and allowed to cool down at ambient temperature in loose condition. The maximum specific gravity of these mixtures (G_{mm}) was measured following Florida's standard test method FM 1-T209. The results are also listed in Appendix N.

4.1.3. Performance Results

Moisture susceptibility, stiffness and resistance to cracking and rutting of high RAP hot recycled mixtures were evaluated to verify adequate performance based on current thresholds for HMA mixtures.

4.1.3.1. Moisture Susceptibility

The moisture susceptibility of hot recycled mixtures was evaluated by means of the modified Lottman test as outlined in FDOT test method FM 1-T283: Resistance of Compacted Bituminous Mixture to Moisture-Induced Damage.

Due to limitations of the SGC equipment to achieve target heights below 1.96 in. (50 mm), three samples of 6 in. (150 mm) diameter were compacted, per hot recycled mixture type, to a target height of 3.1 in. (78.2 mm) and cut in half along the thickness in order to produce six specimens 1.5 in. (38.1 mm) thick.

The total of six specimens were divided in two subsets of three specimens each according to their AV content. The subset of specimens with the lowest AV content was subjected to complete moisture conditioning thorough vacuum saturation followed by freezing at -18°C for 16 hours and thawing in a water bath at 140° F (60°C) for 24 hours. Then ten Hg-in. of partial pressure were applied to each specimen of the subset to vacuum saturate. After this period, the vacuum was removed and the specimens left submerged for five minutes. The other specimen subset was air-conditioned at room temperature during the time required to wet-condition the other subset (approximately 42 hours). Appendix N presents the volumetric properties of the specimens and their degree of saturation.

Both subsets (i.e., unconditioned and moisture conditioned) were tested at the same time, after completing the freeze-thaw conditioning. Indirect tensile (IDT) strength measurements were conducted at room temperature of about 77°F (25°C) under a monotonic load applied at a rate of 2.0 in./minute (50 mm/min), as required by FDOT test method FM 1-T283. It is noteworthy that the moisture conditioned specimens were allowed to reach ambient temperature in a water bath for two hours before testing.

Figure 29 and Figure 30 present the results of IDT strength and TSR obtained for the hot recycled mixtures. Minimum requirements of IDT strength and tensile ratio according to ARRA standard CR 201 are also displayed.

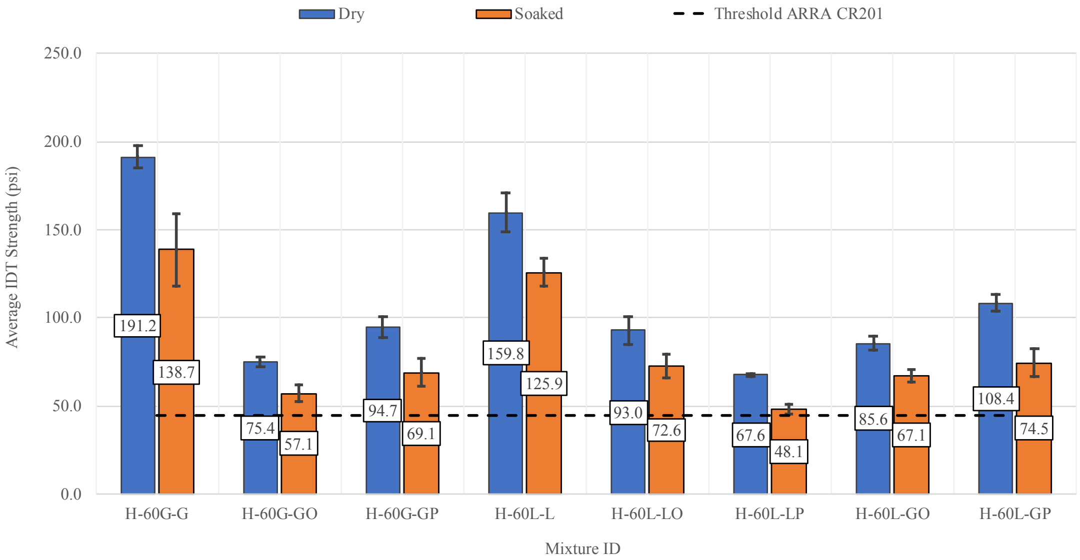


Figure 29. Hot Recycled Mixtures Unconditioned and Moisture Conditioned IDT Strength

The results presented in Figure 29 provide evidence of a general reduction of the mixture IDT strength with the inclusion of recycling agents (both organic and petroleum-based types). This decrease in strength ranges from 32% to 61% for specimens in dry condition and from 41% to 62% for the moisture conditioned specimens. Although the IDT strength of the hot recycled mixtures that included recycling agents was lower than the mixtures without them, the average IDT strength was still above the minimum threshold recommended by ARRA, indicating good performance. Furthermore, the hot recycled mixtures that incorporated granite virgin aggregate and petroleum-based recycling agent (i.e., H-60G-GP and H-60G-GP) developed greater IDT strengths than their counterparts that employed the organic recycling agent. Conversely, the hot recycled mixtures fabricated with limestone virgin aggregate presented a larger IDT strength when the organic recycling agent was used.

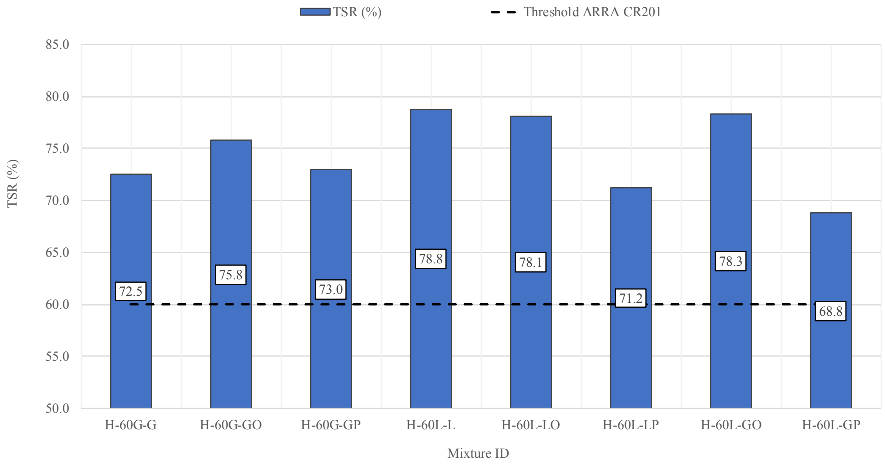


Figure 30. Hot Recycled Mixtures Tensile Strength Ratio

The TSR results presented in Figure 30 are quite homogeneous, ranging from 68.8% to 78.8%. Although all TSR results were above the minimum recommended by ARRA of 60% (given the moisture conditioned IDT strength exceeds the minimum dry strength/stability requirement of 45 psi), suggesting low moisture susceptibility, the effect of the inclusion of recycling agent, the RAP source or virgin aggregate type were not apparent in the results. However, the results did allow identification of lower moisture susceptibility on the order of 3.7% to 12.2% in the recycled mixtures that included the organic-based recycling agent when compared to recycled mixtures with the petroleum-based recycling agent.

4.1.3.2. <u>Rutting & Moisture Susceptibility</u>

The rutting and moisture susceptibility of hot recycled mixtures was evaluated by means of HWTT in accordance with AASHTO T324: *Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA).* As defined by AASHTO T324, the stripping inflexion point (SIP) and rut depth at a certain number of load cycles were determined for each mixture type in order to evaluate moisture susceptibility and rutting resistance, respectively. Two test replicates were concurrently conducted per mixture type employing both wheels of the HWTT equipment (i.e., left and right). Figure 31 and Figure 32 present the SIP obtained on each wheel and the average rut depth versus load cycle, respectively.

According to the results in Figure 31, the determination of the SIP parameter was not possible for most of the hot recycled mixtures in either one or both wheels, indicating that the specimens were resistant to stripping throughout the test.

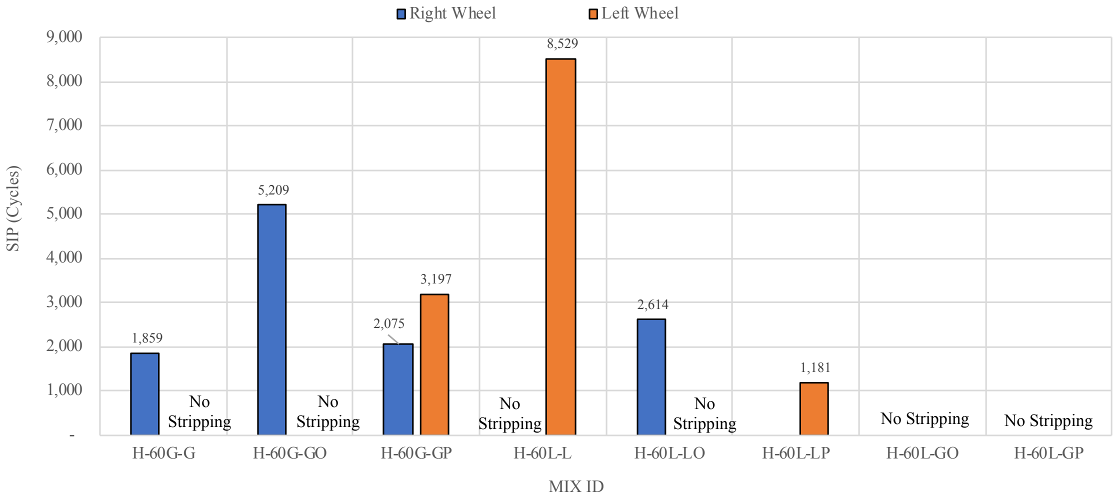


Figure 31. Hot Recycled Mixtures Stripping Inflexion Point (SIP)

From Figure 31, the addition of recycling agents to the mixtures fabricated with limestone/granite RAP and granite virgin aggregate resulted in larger SIP values and hence had greater moisture susceptibility. Likewise, the inclusion of recycling agents on the mixtures fabricated with limestone RAP lowered the SIP and hence increased the moisture susceptibility by about 78%.

Figure 32 presents the average rut depth for each hot recycled mixture type. Most mixtures experienced accelerated rutting at early life. The assigned rut depth failure criteria of ½ in. (12.5 mm) was reached in less than 5,000 load cycles by every mixture fabricated with limestone RAP regardless of the presence or absence of the recycling agent. The mixtures including only virgin aggregate and RAP (i.e., H-60G-G and H-60L-L) exhibited better rutting resistance when compared to equivalent mixtures that included recycling agents.

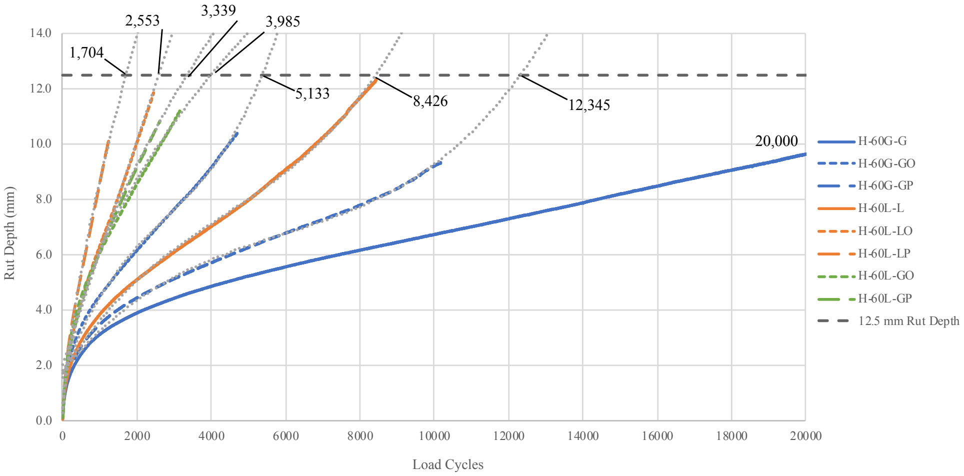


Figure 32. Hot Recycled Mixtures Ruth Depth vs. Load Cycles

In addition to the parameters defined in AASHTO T324, the rutting resistance of the mixtures was evaluated by means of parameters proposed by Yin et al. (2014). To analyze HWTT output, the novel methodology fits a curve of double concavity to rut depth versus load cycles data and assumes that stripping starts at the inflexion of the fitted curve. This point is labeled as striping number (SN). Then, the slope of the fitted curve at the SN ($\Delta\varepsilon^{vp}{}_{SN}$) is the rutting resistance parameter; higher values represent more susceptibility to rutting. Figure 33 presents the $\Delta\varepsilon^{vp}_{SN}$ values for each hot recycled mixture. According to the results, the mixtures prepared with granite virgin aggregate, regardless of the inclusion of recycling agents, exhibited better rutting resistance than the mixtures with limestone virgin aggregate and recycling agents. The rutting susceptibility for mixtures with organic recycling agents exhibited an increase of between 4.2 and 5.7 for limestone and granite aggregates, respectively, as compared to equivalent mixtures without recycling agents. In the case of mixtures with petroleum-based recycling agents, the increase in rutting susceptibility for granite and limestone mixtures as compared to equivalent mixtures without recycling agents was between 1.9 and 7.9, respectively.

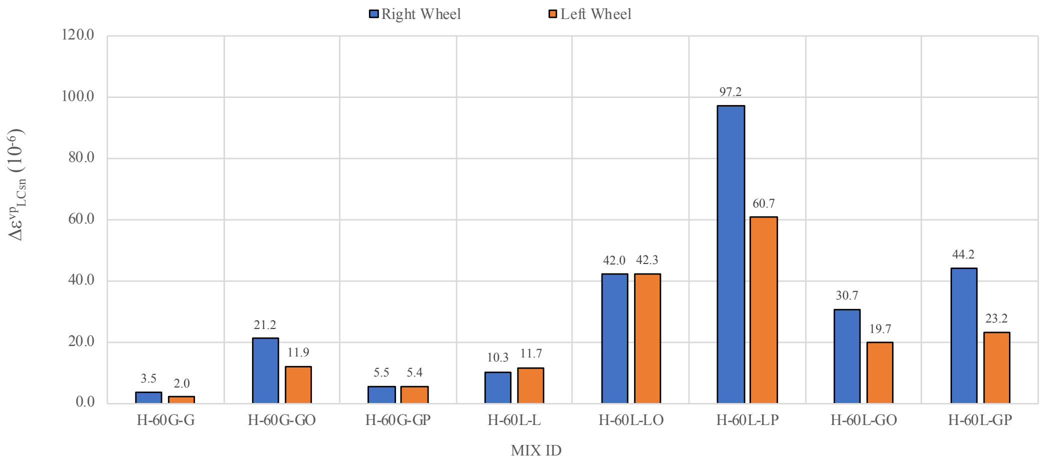


Figure 33. Hot Recycled Mixtures Rutting Resistance Parameter ($\Delta\varepsilon^{vp}_{SN}$)

4.1.3.3. Intermediate Temperature Cracking

The intermediate temperature cracking resistance of the hot recycled mixtures was assessed in accordance with AASHTO TP 124: *Test for Determining the Fracture Potential of Asphalt Mixtures Using Semicircular Bend Geometry (SCB) at Intermediate Temperature.*

Two replicate specimens 6 in. (152.4 mm) diameter were compacted in the SGC to a target height of 1.96 in. (50 mm). As required by the standard test method, each specimen was cut in half and a notch was introduced along the axis of symmetry of the resulting semicircular specimens. Monotonic load was applied until failure at the top of the specimens in a three-point bending arrangement while load and vertical displacement data were recorded during the test.

The cracking resistance of the hot recycled mixtures was characterized by means of the flexibility index (FI) and the cracking resistance index (CRI). The FI was developed by the Illinois Center for Transportation (ICT) and it is defined as the ratio of the work of fracture (i.e., area under the load-displacement curve) by the slope (m) at the post-peak inflexion point of the curve fitted to the load versus displacement data. High FI values suggest better cracking resistance of the mixture. Nevertheless, the FI is not always able to properly characterize the cracking behavior of brittle mixtures (i.e., mixtures with large amounts of RAP), where instantaneous failures hinder the measurement of load and displacement after the peak load, and hence, the ability to compute the slope m. Ongoing NCHRP 09-58 (Kaseer et al., 2018) project developed the CRI to overcome the brittle failure issue. CRI is calculated as the fracture energy up to the peak load, divided by the magnitude of the peak load registered during the test.

Figure 34 and Figure 35 present the average and standard deviation of the FI and CRI respectively, for the hot recycled mixtures. Both FI and CRI seemed to agree in the characterization of the cracking behavior of the hot recycled mixtures, since both parameters displayed similar trends when varying the recycling agent and virgin aggregate type. Although discrepancies are observed for the H-60L-LO and H-60L-LP mixtures where CRI suggest a better cracking performance for the latter, the differences were negligible.

The mixtures with no recycling agent (i.e., H-60G-G and H-60L-L) presented the lowest cracking indices, which agrees with the IDT strength results (Figure 29) and suggest stiffer less ductile binders in the mixtures. Conversely, the mixtures in which the rheology of the recycled binder was intended to be restored by recycling agents, displayed improved intermediate cracking behavior (i.e., greater FI and CRI values). Therefore, the inclusion of recycling agents seems to help in the control of the cracking performance of stiff recycled mixtures.

The mixtures that included recycling agents improved their FI with respect to equivalent mixtures without recycling agents from 45% to 145% for granite virgin aggregate mixtures and around 160% for limestone virgin aggregate mixtures. Likewise, the improvement in the CRI ranges from 28% to 61% for the granite virgin aggregate mixtures and around 50% for the limestone virgin aggregate mixtures.

The results also show that the mixtures including limestone RAP, regardless of the recycling agent type or virgin aggregate type, reached approximately the same FI and CRI values, whereas the mixtures including limestone/granite RAP and granite virgin aggregate exhibited better cracking performance (i.e. greater FI and CRI) when including the petroleum-based recycling agent.

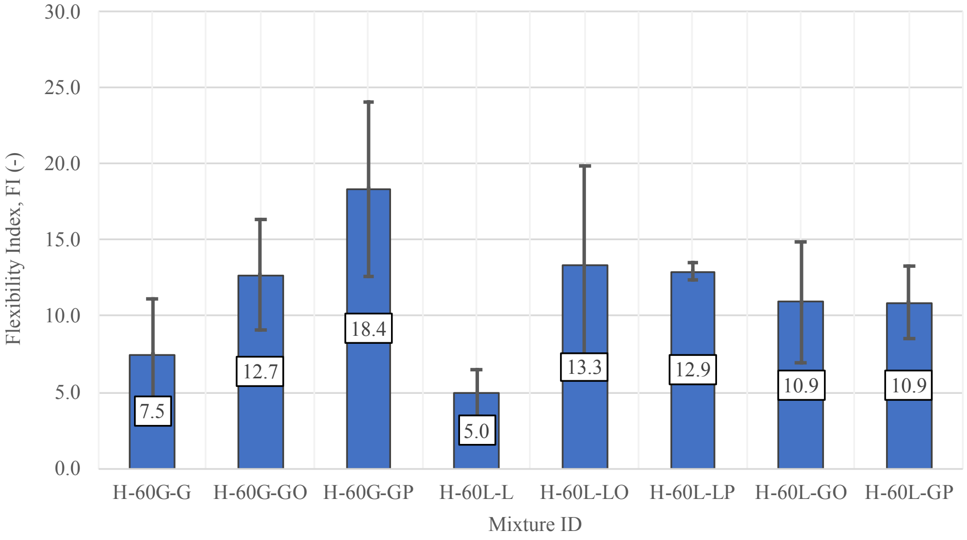


Figure 34. Hot Recycled Mixtures Flexibility Index (FI)

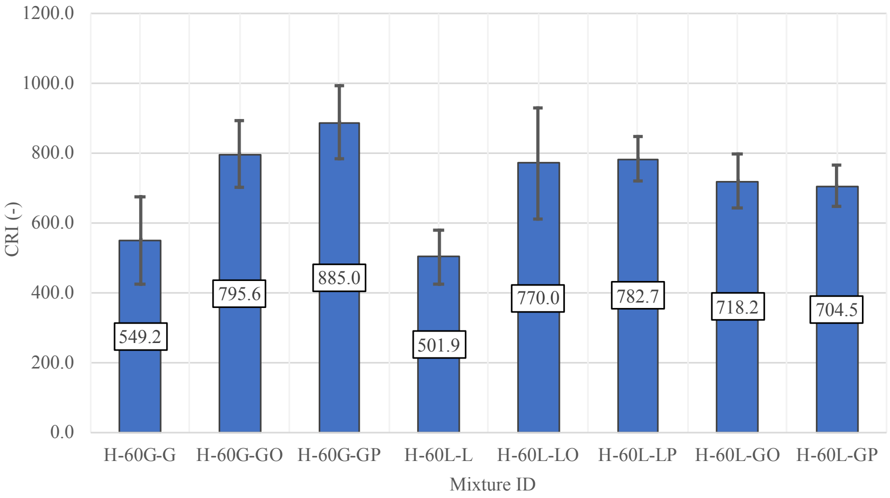


Figure 35. Hot Recycled Mixtures Cracking Resistance Index

4.1.3.4. Stiffness

The stiffness of the hot recycled mixtures was evaluated by conducting the resilient modulus (M_R) test determined in accordance with ASTM D7369: *Standard Test Method for Determining the Resilient Modulus of Bituminous Mixtures by Indirect Tension Test.* Given the nondestructive nature of the test, M_R measurements were conducted on specimens destined to conduct the intermediate temperature cracking evaluation (i.e., SCB) prior to cutting and notching of the samples. Therefore, M_R measurements per hot recycled mixture type were conducted on three specimens 6 in. (152.4 mm) diameter by 1.96 in. (50 mm) height. In order to calculate the M_R, a Poisson's ratio of 0.35 was assumed based on the test temperature (i.e., 77°F [25°C]). After conditioning, a repetitive haversine compressive load pulse was applied in the vertical diametral plane of the specimens and the horizontal deformation registered through a set of two LVDTs aligned along the diametral plane.

Figure 36 present the average and standard deviation of the M_R measurements for each type of hot recycled mixture. Similarly to the IDT strength result, the M_R values for the mixtures with recycling agents was compared to the mixtures with no recycling agents. The reduction in stiffness between these two groups of mixtures was between 36% and 57% for the granite virgin aggregate mixtures and from 46% and 60% for the limestone virgin aggregate mixtures. Furthermore, the mixtures including granite virgin aggregate and the petroleum-based recycling agent developed slightly higher levels of stiffness than equivalent mixtures prepared with the organic recycling agent. This was the case for the H-60G-GP and H-60L-GP mixtures. Conversely, the mixture fabricated with limestone virgin aggregate and the organic recycling agent presented a larger M_R value as compared to its counterpart prepared with the petroleum-based recycling agent.

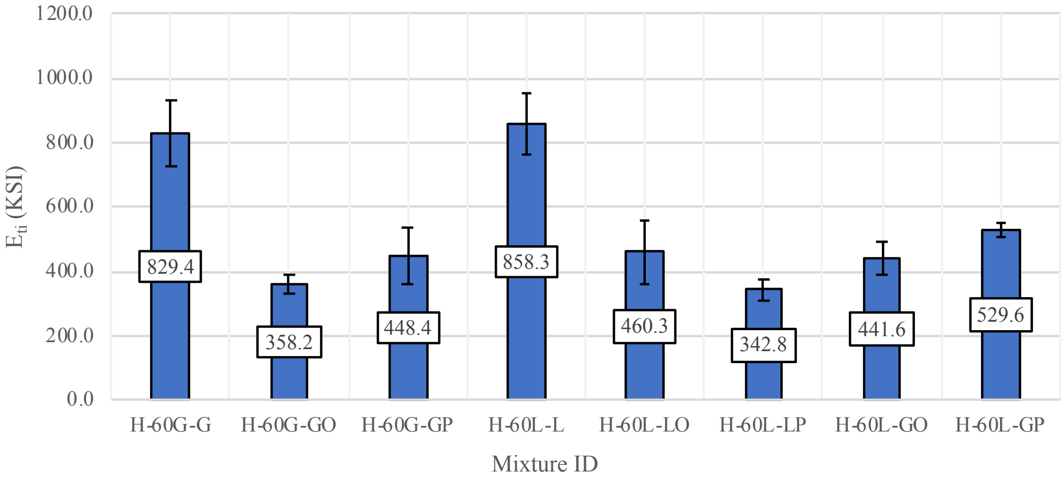


Figure 36. Hot Recycled Mixtures Resilient Modulus (M_R)

4.2. COLD RECYCLED MIXTURES – EMULSIFIED ASPHALT

The materials employed in the production of the emulsified cold recycled asphalt mixtures correspond to the RAP sources (Stockpile 1-09 and Stockpile 1-16) and virgin aggregates types (limestone and granite) shown in Table 5.

Additionally, instead of binder PG 52-28, a slow setting cationic emulsion of low viscosity and hard asphalt residue coded CSS-1H (TxDOT, 2015) was employed for the design and production of the recycled mixtures. The selection of the emulsion product was based on input from industry representatives when consulted about the popular types of emulsions used for cold recycling. Table 24 presents the emulsion properties as reported in the Safety Data Sheet.

Table 24. CSS-1H Emulsion Properties

Chemical Name	%
Asphalt	50 - 70
Water	30 - < 40
Hydrochloric Acid	< 2

This section describes the procedure followed for the design of the emulsified cold recycled asphalt mixtures, specimen preparation and performance results.

4.2.1. Mixture Design

The following sections present the steps followed for the design of cold recycled mixtures with emulsified asphalt: (1) aggregate blend proportioning, (2) optimum moisture content determination and (3) optimum emulsion content determination.

Three combinations of aggregate type and RAP source, hereafter referred to as aggregate blends, were selected for the design of emulsified cold recycled mixtures (Table 25).

4.2.1.1. Material Proportioning

Considering the nature of cold recycled asphalt mixtures, base materials specifications were used rather than HMA specifications in order to establish the design aggregate gradations. Therefore, FDOT Standard Specification, Section 234: *Superpave Asphalt Base* was employed to determine the aggregate blend proportioning.

For each aggregate blend, #78 intermediate size stone and W-10 fine screenings were blended with RAP to meet the aggregate gradation requirements established in FDOT Standard Specification, Section 234-1: *Description*. For the design of the cold recycled mixtures, the RAP was considered as a “black rock”; that is, it was assumed that the binder coating the RAP particles did not activate during mixing. Therefore, the gradations of the RAP before ignition oven (i.e., including the asphalt coating the rock) were determined following the standard test method AASHTO T-27 (Appendix C) and were employed to meet the gradation requirements.

Only one type of asphalt base is defined in FDOT Standard Specification, Section 234: base with NMAS of ½ in. (12.5 mm) (i.e., B-12.5). The base types were extended to include a NMAS of ¾ in. (19.0 mm) (i.e., B-19.0), making an allowance to accommodate the larger particle sizes observed in the limestone intermediate stone (C-41) and limestone/granite RAP (Stockpile 1-16). Gradation requirements for each NMAS gradation are shown in Appendix C.

The aggregate blends proportions of the #78 stone, W-10 screenings and RAP that met the gradation requirements are shown in Table 25. Figure 37 through Figure 39 show the resulting aggregate gradation curves for each aggregate blend.

Table 25. Emulsified Cold Recycled Aggregate Blends Proportions

Aggregate's blend	RAP			Virgin Aggregate		
	Source	Aggregate Type	Amount (%)	Type	#78 Stone (%)	W-10 Screenings (%)
ABC-100L-E	Stockpile 1-09	Limestone	100	-	-	-
ABC-60L-LE	Stockpile 1-09	Limestone	60	Limestone	25	15
ABC-60G-GE	Stockpile 1-16	Limestone /Granite	60	Granite	5	35

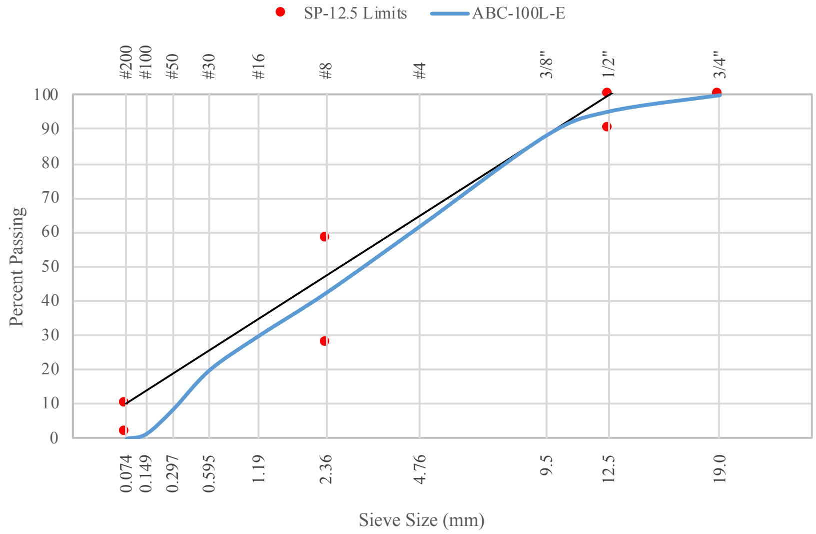


Figure 37. Emulsified Cold Recycled Mixtures ABC-100L-E Aggregate Blend Gradation Curve

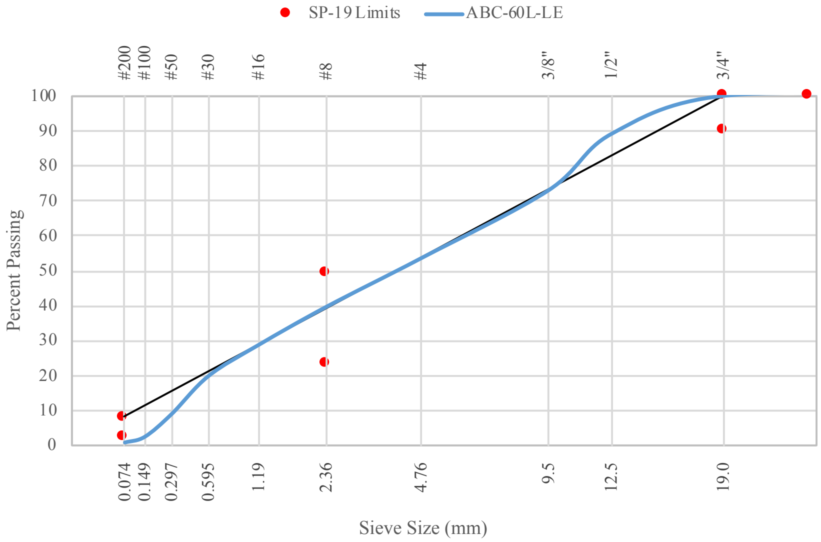


Figure 38. Emulsified Cold Recycled Mixtures ABC-60L-LE Aggregate Blend Gradation Curve

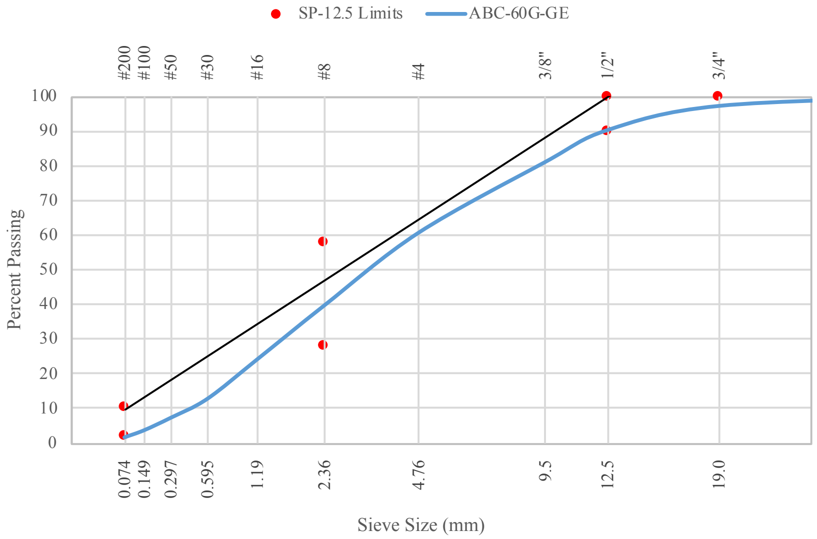


Figure 39. Emulsified Cold Recycled Mixture ABC-60G-GE Aggregate Blend Gradation Curve

The aggregate blend ABC-100L-E and ABC-60G-GE resulted in a NMAS of ½ in. (12.5 mm), whereas ABC-60L-LE blend resulted in a NMAS of ¾ in. (19.0 mm). Therefore, the mixtures produced and requirements verified for the ABC-100L-E and ABC-60G-GE aggregate blends corresponded to B-12.5 and for the ABC-60L-LE aggregate blend to B-19.0. Detailed aggregate blend calculations are shown in Appendix C.

4.2.1.2. Optimum Moisture Content Determination

The OMC of the aggregate blends presented in Table 25 was defined as the required added moisture for the production of the emulsified cold recycled asphalt mixtures. Moisture-density curves were established for the ABC-100L-E and ABC-60L-LE blends following the Florida test method FM 1-T180: *Moisture-density relations of soils using a 4.54-kg [10-lb] rammer and a 457-mm [18-in.] drop*. Figure 40 presents the results.

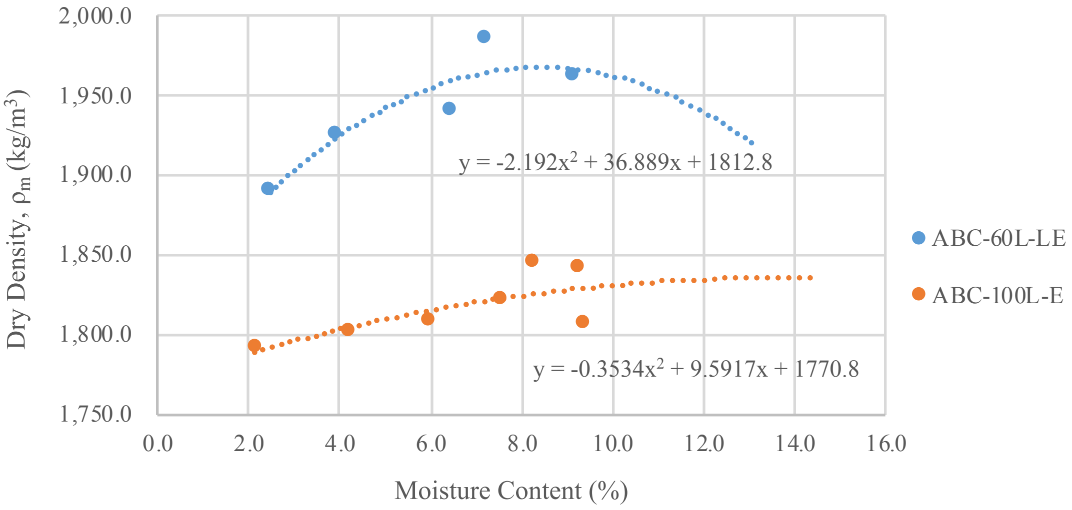


Figure 40. Aggregate Blends Moisture-Density Curves

The OMC is obtained from the moisture-density curves as the point where maximum density is obtained. Using the regression curves, the resulting OMC was 13.6% for the ABC-100L-E blend and 8.4% for the ABC-60L-LE blend. However, after attempting to fabricate specimens using these OMC values, the resulting specimens had excessive water as shown in Figure 41.


(a) (b)

Figure 41. Cold Recycled Asphalt Mixtures; (a) Loose Mixture OMC=8%, (b) Compacted Specimen OMC=8%

Based on past research presented previously in the literature review, a reduction to the MC at mixing was considered necessary. Mamlouk and Ayoub (1983), Scholz et al.(1991), Khosla and Bienvenu (1996) and Kim et al. (2011) employed fixed MC values ranging from 1% to 5%. Babei, Walter (1989) and Kim et al. (2011) defined an MC limit of 4% for proper compaction. Cox et al. (2015) found an average MC of 3.5% in 43 CIR projects. Considering these research results, normal practices in the literature and states' DOTs recommendations, the OMC for the production of cold recycled mixtures was reduced to 4.0%.

4.2.1.3. Optimum Emulsion Content

Cold recycled mixtures with three emulsion contents were fabricated for each aggregate blend shown in Table 25 in order to find the optimum emulsion content (OEC) that satisfied the minimum indirect tensile strength requirement specified in ARRA standard CR 201.

According to Table 24, the asphalt proportion (AP) of the CSS-1H emulsion is 60%. Thus, the actual amount of binder added to the emulsified cold recycled mixture, also known as residual binder content (RBC), was estimated through Equation 5.

$$RBC = \frac{EC}{AP_{Emulsion}} \qquad \textbf{Equation 5}$$

Before mixing, the aggregates (#78 and W-10) and RAP were dried overnight and for four hours, respectively, in an oven at 230ºF (110°C). The materials were allowed to cool down to room temperature and then mixed with the 4.0% OMC and the target EC employing a mechanical mixer.

Once mixing was complete, four specimens of 6 in. (152.4 mm) diameter by approximately 1.5 in. (38.1 mm) height were compacted, per each EC in the SGC to N_{Design} = 30 gyrations as established in ARRA standard CR 201.

Curing Protocol

The curing time of the compacted cold recycled asphalt mixture specimens was determined prior to production. As outlined in the ARRA standard CR 201, test specimens were cured in a forced draft oven at 140ºF (60°C) until constant weight was achieved (i.e., 0.05% max change in weight in two hours).

The effect of RAP and EC on the curing time was evaluated in the protocol experiment. Four 6 in. (152.4 mm) diameter by approximately 1.5 in. (38.1 mm) height specimens of the ABC-100L-E and ABC-60L-LE aggregate blends were fabricated with an EC of 6.5% (3.9% RBC). An additional four specimens of the ABC-100L-E blend were fabricated with an EC of 8.0% (4.8% RBC). All mixtures were fabricated employing the defined OMC of 4.0%.

Figure 42 presents the evolution in time of the average weight change for each of the test mixtures produced. The results showed that both of the C-100L-E mixtures (6.5% and 8.0% EC) present a weight stabilization after approximately 25 hours of curing and the C-60L-LE mixture after 20 hours. Detailed measurements of weight loss for every specimen and mixture are presented in Appendix L. Based on the experiment results, a curing period of 24 hours at a temperature of 140°F (60°C) was selected for all aggregate blends. After curing, the specimens were allowed to cool down, for at least 12 hours, on a flat surface.

Table 26 summarizes the emulsified cold recycled mixtures OMC and selected curing time determined using the aggregate blends listed in Table 25.

Table 26. Emulsified Cold Recycled Mixtures OMC and Curing Time

Cold Recycled Mixture ID	Aggregate Blend	OMC	Curing Time @ 60ºC
C-100L-E	ABC-100L-E	4%	24h
C-60L-LE	ABC-60L-LE		
C-60G-GE	ABC-60G-GE		

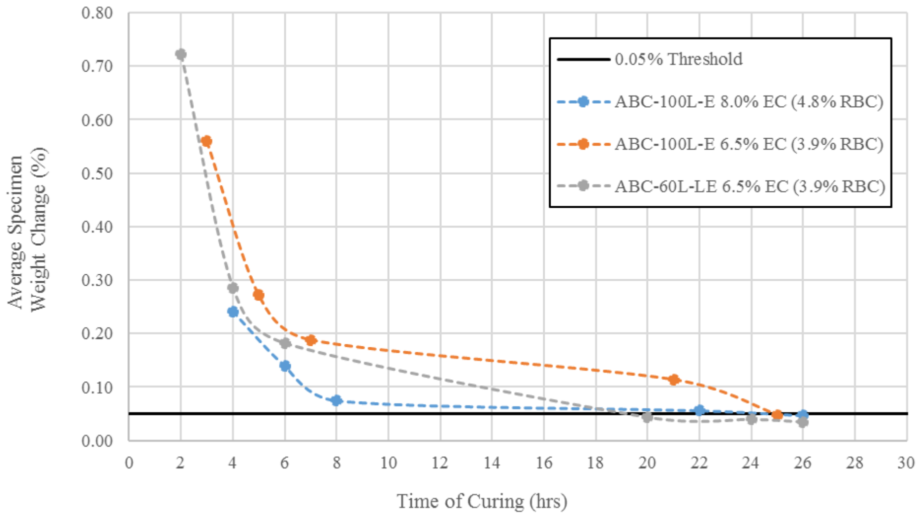


Figure 42. Curing Protocol Experiment Average Specimen Weight Loss

IDT Strength Results

Two specimens per EC were moisture conditioned in a water bath at room temperature for 24 hours. Two other compacted specimens of the same EC were tested without conditioning. The IDT strength was determined in accordance with FM 1-T283 for both dry and moisture conditioned specimens.

Figure 43 through Figure 45 present the unconditioned and moisture conditioned IDT strength. A minimum indirect tensile strength threshold of 45 psi (310 kPa) is indicated by ARRA standard CR 201, Table 1. This value was used to select the optimum EC.

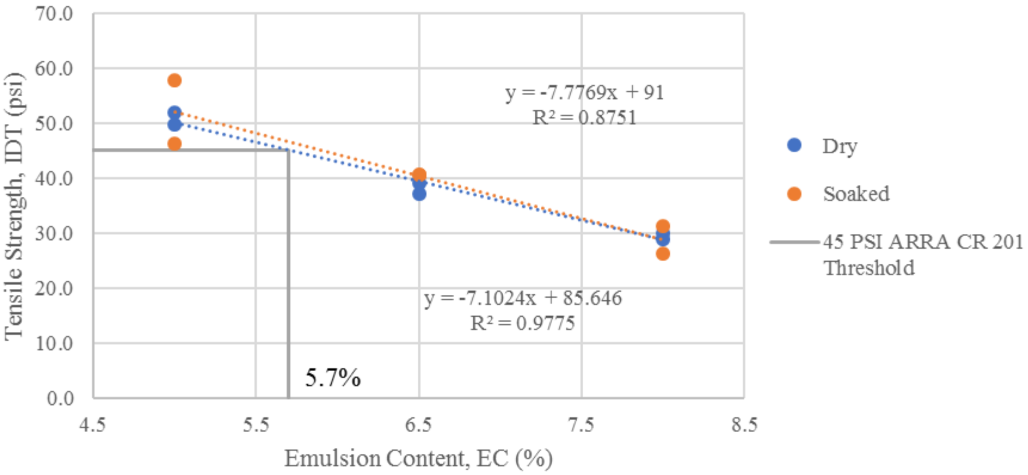


Figure 43. C-100L-E Emulsified Cold Recycled Mixture IDT Strength

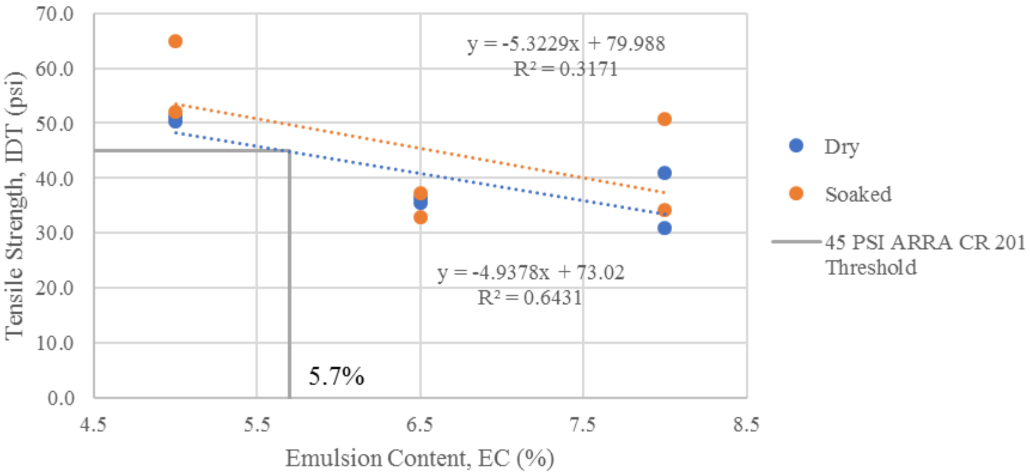


Figure 44. C-60L-LE Emulsified Cold Recycled Mixture IDT Strength

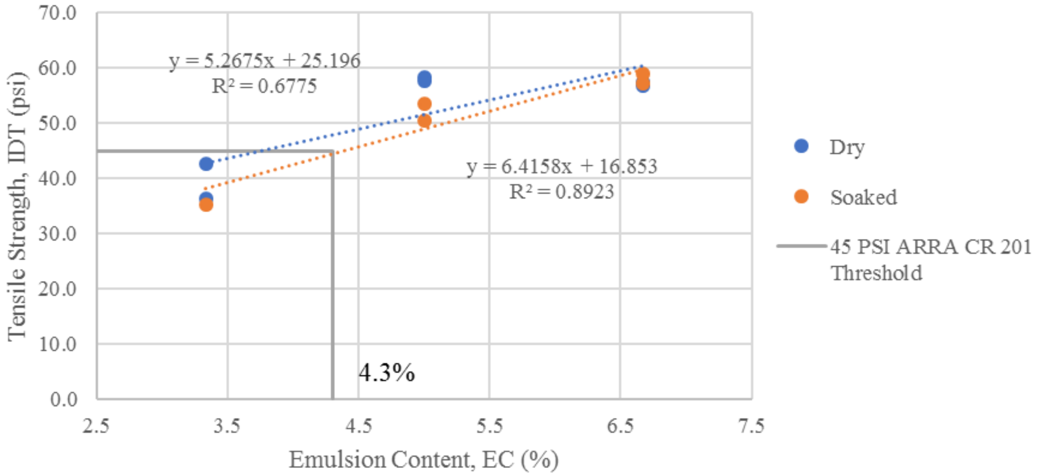


Figure 45. C-60G-GE Emulsified Cold Recycled Mixture IDT Strength

Table 27 and Table 28 present the TSR for each mixture and EC. A minimum TSR of 70% is defined in ARRA standard CR 201, Table 1 for mixtures that incorporate RAP, with a provision to reduce TSR to 60% if the IDT strength of the moisture conditioned specimens exceeds the minimum dry strength/stability requirement of 45 psi.

Table 27. C-100L-E and C-60L-LE Mixtures TSR Results

EC (RBC) (%)	Tensile Strength Ratio, TSR (%)	
	C-100L-E	C-60L-LE
5.0 (3.0)	100	120
6.5 (3.9)	110	100
8.0 (4.8)	100	120

Table 28. C-60G-GE Mixture TSR Results

EC (RBC) (%)	Tensile Strength Ratio, TSR (%)
	C-60G-GE
3.3 (2.0)	90
5.0 (3.0)	90
6.7 (4.0)	100

Table 29 presents the OEC and corresponding TSR interpolated from the data presented in Table 27 and Table 28. Based on these TSR values, the mixtures did not exhibit moisture susceptibility, hence no stabilization by means of addition of lime was considered necessary.

Table 29. Optimum Emulsion Content

Cold Recycled Mixture	OEC (ORBC) (%)	TSR (%)
C-100L-E	5.7 (3.4)	100
C-60L-LE	5.7 (3.4)	110
C-60G-GE	4.3 (2.6)	90

4.2.2. Specimen Fabrication

Specimens for six types of emulsified cold recycled mixtures were fabricated. The proportion of aggregate and RAP determined to meet FDOT Standard Specification, Section 234 are presented in Table 30. The OEC employed in the mixtures production was defined by pairing the virgin aggregate type of the mixture designs presented in Table 29, with the type of virgin aggregate in each mixture (Table 30). Since no variation in OEC was observed with respect to the amount of RAP included in the mixture, the OEC was equivalent for every mixture with the same virgin aggregate type regardless of the RAP content. Therefore, mixtures with limestone virgin aggregate were assigned an OEC of 5.7% (3.4% ORBC), while mixtures with granite virgin aggregate were assigned an OEC of 4.3% (2.6% ORBC).

Table 30. Emulsified Cold Recycled Mixtures Material Proportions

Mixture Type	RAP Type and Amount		Virgin Aggregate Type Proportioning				OEC (ORBC) (%)	OMC (%)
			Limestone		Granite			
	STK 09	STK 16	#78 Stone	W-10 Screenings	#78 Stone	W-10 Screenings		
C-60L-LE	60.0%	-	25.0%	15.0%	-	-	5.7 (3.4)	4.0
C-80L-LE	80.0%	-	20.0%	0.0%	-	-	5.7 (3.4)	4.0
C-100L-E	100.0%	-	-	-	-	-	5.7 (3.4)	4.0
C-60G-GE	-	60.0%	-	-	5.0%	35.0%	4.3 (2.6)	4.0
C-80G-GE	-	80.0%	-	-	0.0%	20.0%	4.3 (2.6)	4.0
C-60L-GE	60.0%	-	-	-	20.0%	20.0%	4.3 (2.6)	4.0

Table 31 presents the list of the tests employed in the performance evaluation of recycled mixtures with emulsified asphalt. Specimen characteristics and quantities is also provided. A total of 78 specimens were fabricated.

Table 31. Emulsified Cold Recycled Mixture Specimen Characteristics and Quantities

Mixture Property	Test	Standard	Diameter, in. (mm)	Compaction Criteria	Number of Replicates per Mixture Type	Total Number of Specimens
Moisture Susceptibility	Modified Lottman, Indirect Tension Test (IDT)	FM 1-T283	6 (152.4)	N_{design}=30 gyrations	6	36
Rutting & Moisture Susceptibility Hamburg Wheel	Hamburg Wheel Track Test (HWTT)	AASHTO T 324	6 (152.4)	Height: 2.5 in. (63.5 mm)	4	24
Stiffness	Resilient Modulus (M_R)	ASTM D7369	6 (152.4)	Height: 2.5 in. (63.5 mm)	*	*
Durability	Cantabro	AASHTO TP 108	6 (152.4)	Height: 4.5 in (115.0 mm)	3	18

*The M_R test was conducted on three of the four specimens fabricated for rutting and moisture susceptibility evaluation.

Before mixing, the virgin aggregates (#78 and W-10) and RAP were oven dried overnight and for four hours, respectively, at 230ºF (110°C). The materials were allowed to cool down to room temperature and then mixed with the 4.0% OMC and the OEC employing a mechanical mixer.

Once mixing was complete, the specimen replicates listed in Table 31 were compacted in the SGC using the compaction criteria that is also listed in Table 31. After compacting, the specimens cured for 24 hours in a forced draft oven at 140ºF (60ºC). Next, the specimens were taken out of the oven and placed on a flat surface to cool down for at least 24 hours before testing. After the cool down period, the mass of the specimen in air, the mass of the specimen soaked in water and the mass in saturated surface dry (SSD) condition was determined as required by Florida's standard test method FM 1-T166. Appendix N presents the bulk specific gravity (G_{sb}) and estimated AV content for each specimen and mixture defined in Table 30 and Table 31.

Two additional samples were fabricated for each cold recycled asphalt mixture displayed in Table 29 and allowed to cool down at ambient temperature in loose condition. The maximum specific gravity of these mixtures (G_{mm}) was estimated following Florida's standard test method FM 1-T209. The results are also listed in Appendix N.

4.2.3. Performance Results

Moisture susceptibility, rutting, durability and stiffness of the emulsified cold recycled mixtures were evaluated to verify adequate performance based on current thresholds for cold recycled mixtures.

4.2.3.1. Moisture Susceptibility

The moisture susceptibility of cold recycled mixtures with emulsified asphalt was evaluated by means of the modified Lottman test as outlined in FDOT test method FM 1-T283.

Prior to the moisture susceptibility evaluation, moisture conditioning trials were performed on compacted specimens subjected to complete and reduced moisture conditioning protocols. The complete freeze-thaw conditioning protocol as prescribed in FM 1-T283 consisted of vacuum saturation, freezing at -18°C for 16 hours and thawing in a water bath at 140° F (60°C) for 24 hours. The reduced moisture conditioning protocol consisted of vacuum saturation followed by a 24-hour water bath at room temperature. The latter procedure corresponds to the moisture conditioning procedure recommended by ARRA standard CR 201 plus vacuum saturation. Figure 46 compares the IDT strengths resulting from both conditioning protocols.

The results show that the complete moisture conditioning protocol resulted in significantly lower IDT strengths as compared to the specimens subjected to the reduced moisture conditioning protocol. The difference in IDT strengths were between 16% and 73%. Therefore, the moisture conditioning protocol currently prescribed for HMA mixtures in FM 1-T283 was considered too severe for the emulsified cold recycled mixtures and vacuum saturation plus a 24-hour water bath at room temperature was used instead for the moisture susceptibility evaluation.

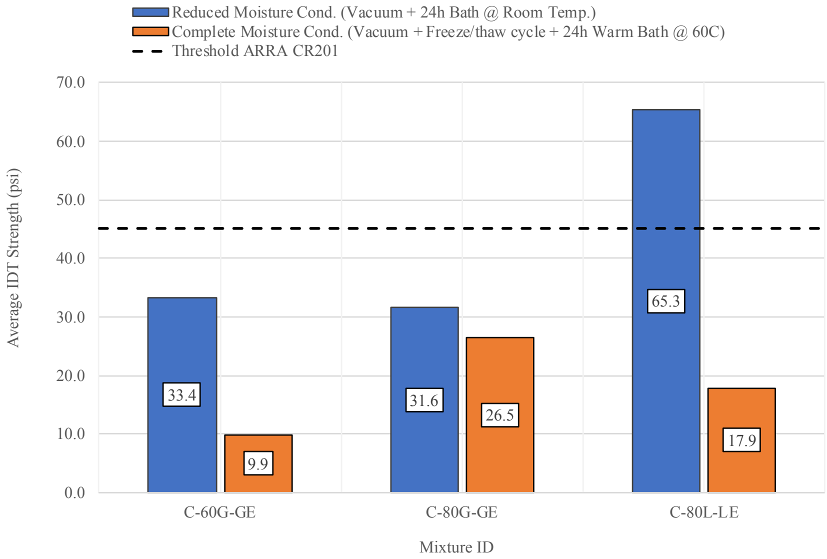


Figure 46. Emulsified Cold Recycled Mixtures IDT Strength for Specimens Subjected to Different Moisture Conditioning Protocols

The six replicate specimens per emulsified cold recycled mixture type were divided into two subsets of three specimens each according to their AV content. The subset with the lowest AV content was moisture conditioned using the reduced moisture susceptibility protocol. The other subset was air-conditioned at room temperature throughout the time required to moisture condition the other subset. Appendix N presents the volumetric properties and the vacuum saturation level achieved. Both subsets were tested at the same time, after the moisture conditioning was completed. IDT strength measurements were conducted at room temperature (77°F [25°]) under a monotonic load applied at a rate of 2.0 in./min (50 mm/min), as required by FM 1-T283.

Figure 47 and Figure 48 present the IDT strength and TSR results obtained for the emulsified cold recycled mixtures. Minimum requirements of IDT strength and TSR according to ARRA standard CR 201 are also displayed.

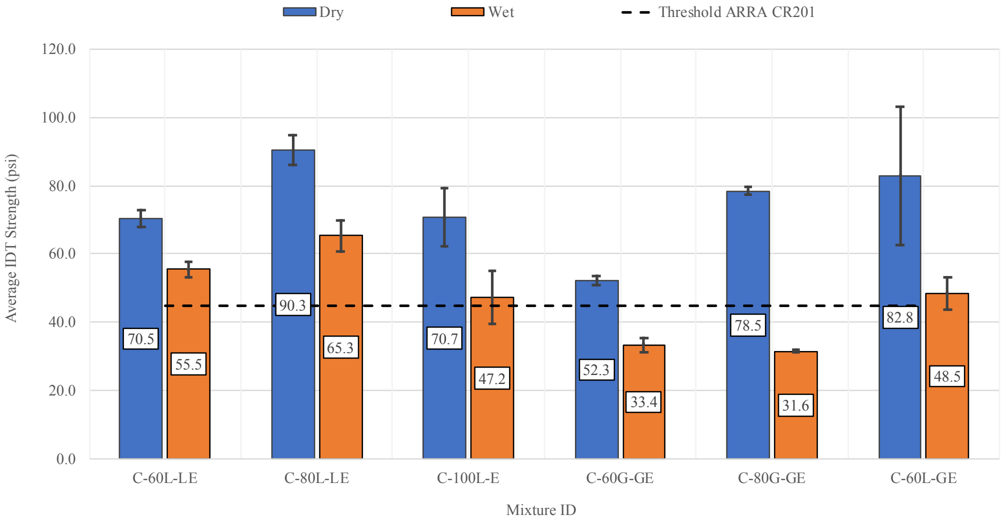


Figure 47. Emulsified Cold Recycled Mixtures IDT Strength Results

According to Figure 47, the emulsified cold recycled mixtures fabricated with limestone RAP exhibited good IDT strength performance. Regardless of the RAP content, the average IDT strength of these mixtures were found to meet the minimum IDT strength requirement recommended by ARRA. In contrast, the unconditioned specimens fabricated with limestone/granite RAP had adequate performance, but failed to pass the minimum IDT strength threshold after moisture conditioning. Taking into account the variability presented by the C-60L-GE mixture, the largest IDT strengths were achieved by the emulsified cold recycled mixtures with 80% RAP content.

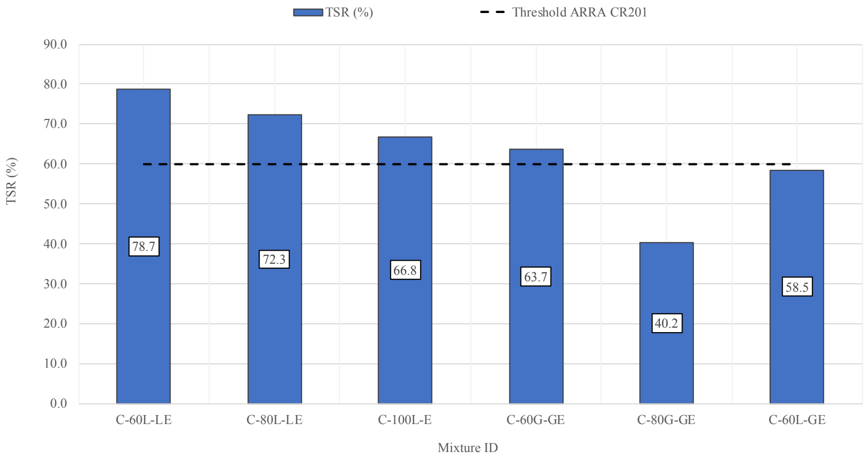


Figure 48. Emulsified Cold Recycled Mixtures TSR Results

From Figure 48, the TSR results of the mixtures developed an inverse relationship with respect to the RAP content. This means that mixtures with higher RAP content developed lower IDT strengths after moisture conditioning, and hence, were more susceptible to induced moisture damage. The moisture susceptibility performance of mixtures including limestone RAP and limestone virgin aggregate could be considered adequate since the minimum TSR requirement was met. However, the mixtures fabricated with more than 60% limestone/granite RAP and granite virgin aggregate failed to meet the minimum TSR requirements due to considerable reduction of the tensile strength after moisture conditioning the specimens.

4.2.3.2. Rutting & Moisture Susceptibility

The moisture susceptibility of the emulsified cold recycled mixtures was evaluated by means of the HWTT in accordance with AASHTO T 324. The SIP and rut depth at a certain number of load cycles were determined for each mixture in order to evaluate moisture susceptibility and rutting resistance, respectively. Two test replicates were concurrently conducted per mixture type employing both wheels of the HWTT equipment (i.e., left and right). Figure 49 and Figure 50 present the SIP obtained on each wheel and the average rut depth versus load cycles, respectively.

According to the results shown in Figure 49, the determination of the SIP parameter was not feasible for two of the emulsified cold recycled mixtures in either one or both wheels, which indicates that the test specimens did not exhibit stripping throughout the test.

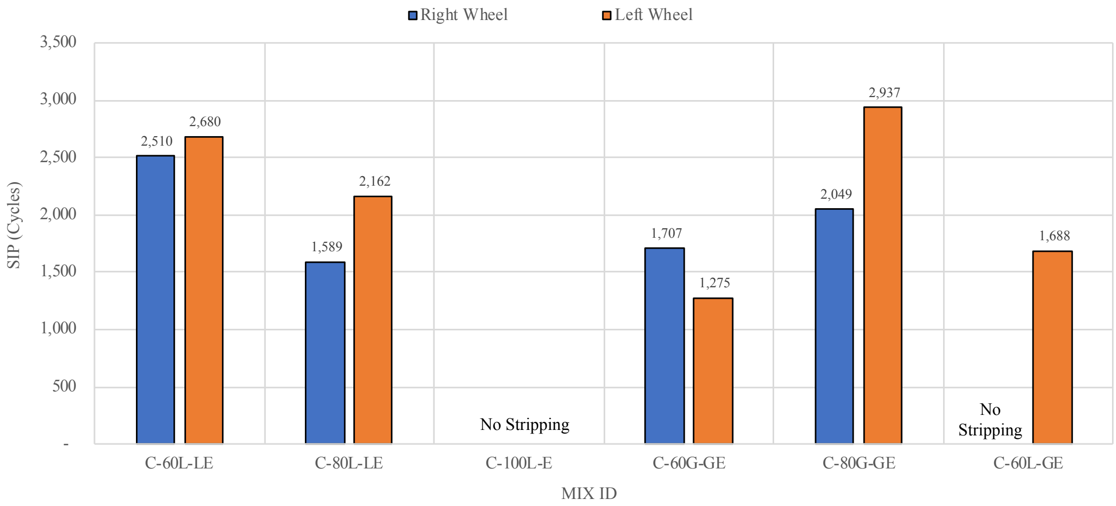


Figure 49. Emulsified Cold Recycled Mixtures Stripping Inflexion Point (SIP)

From Figure 49 it is also possible to observe that increasing the limestone RAP content of mixtures fabricated with limestone virgin aggregate resulted in a reduction of the SIP and hence an increase in moisture susceptibility. Conversely, increasing the limestone/granite RAP content of mixtures fabricated with granite virgin aggregate increased the SIP, resulting in improved moisture susceptibility. Mixture C-100L-E and C-60L-GE did not present strong evidence of stripping.

Figure 50 presents the average rut depth of each emulsified cold recycled mixture. All specimens experienced accelerated rutting at early test stages. The assigned rut depth failure criteria of ½ in. (12.5 mm) was reached in less than 5,000 load cycles by all mixtures regardless of the RAP and virgin aggregate type and content. Mixtures C-60L-LE and C-80G-GE exhibited better rutting performance.

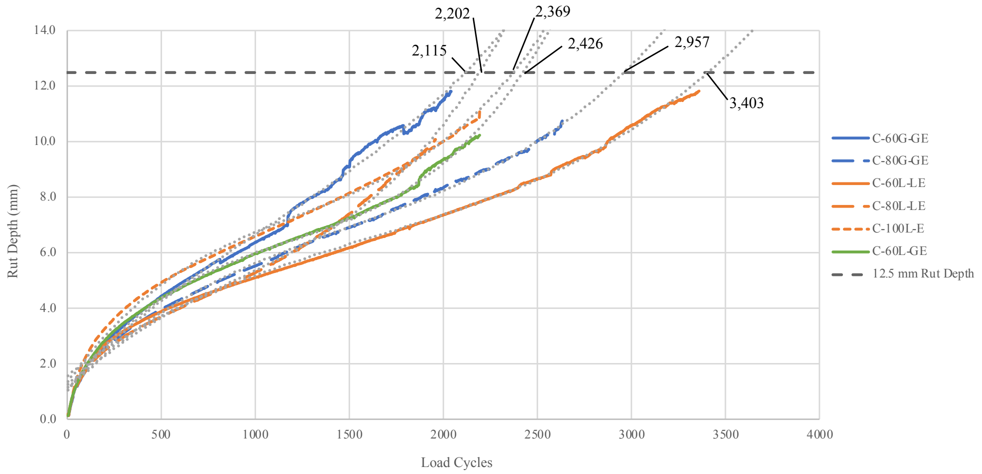


Figure 50. Emulsified Cold Recycled Mixtures Ruth Depth vs. Load Cycles

Figure 51 presents the $\Delta\varepsilon^{vp}_{SN}$ values for each emulsified cold recycled mixture. Mixtures including limestone and virgin aggregate with limestone RAP exhibited lower rutting resistance parameter values at RAP contents of 60 and 100%. Conversely, mixtures with limestone/granite RAP presented better rutting resistance parameter values at a RAP content of 80%. However, it is noteworthy that there is a significant amount of variability between replicates, yielding a wide range of rutting resistance parameter values, ranging from 23.5 to 59.8.

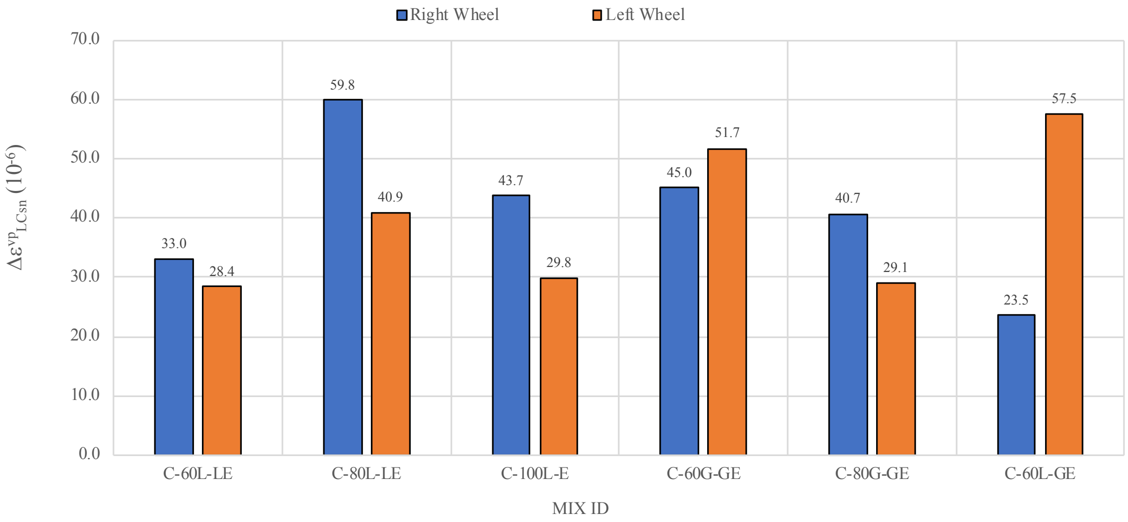


Figure 51. Emulsified Cold Recycled Mixtures Rutting Resistance Parameter ($\Delta\varepsilon^{vp}_{SN}$)

4.2.3.3. Durability

The durability of the emulsified cold recycled mixtures was assessed with the Cantabro abrasion loss test in accordance with AASHTO TP 108: *Standard Method of Test for Abrasion Loss of Asphalt Mixture Specimens.*

Figure 52 presents the average mass loss of compacted specimens after conducting the test. Three replicates per emulsified cold recycled mixture type were conducted. The minimum requirement for adequate raveling performance is also displayed.

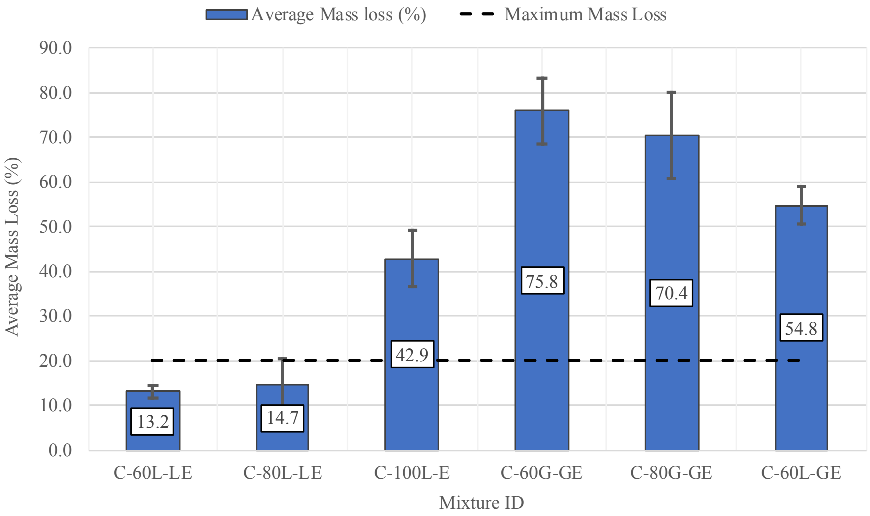


Figure 52. Emulsified Cold Recycled Mixtures Cantabro Abrasion Mass Loss

The mixtures fabricated with granite virgin aggregate, regardless of the RAP content and type, exhibited high mass loss, from 55% up to 76%. Conversely, mixtures fabricated with limestone RAP and limestone virgin aggregate presented, for RAP contents up to 80%, good durability with mass loss values of 15% or less. However, the mixture with only limestone RAP (C-100L-E), when compared to the other specimens that also had 60 and 80% limestone RAP contents, presented a much larger mass loss of 43%. Figure 53 shows how the test specimens looked before and after conducting the Cantabro abrasion loss test.

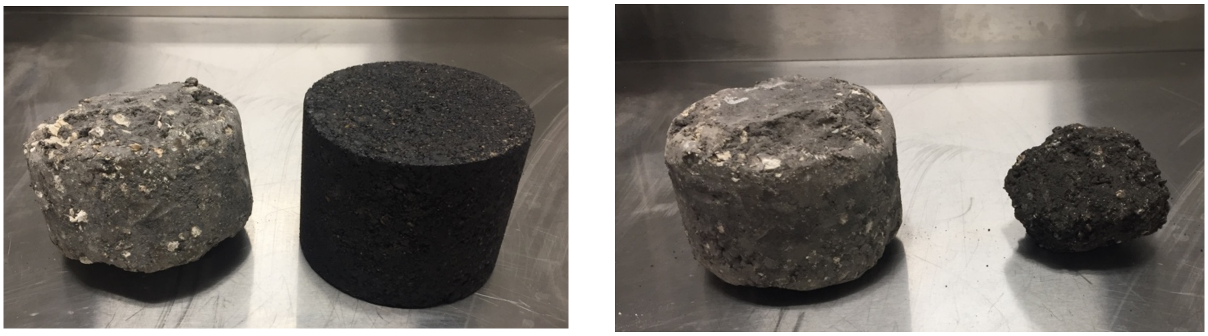

(a) (b)

Figure 53. (a) Specimens Before and After Cantabro Abrasion Loss Test Mixture C-60L-LE and (b) Specimens of Mixture C-60G-GE compared to C-60L-LE

4.2.3.4. Stiffness

The stiffness of the cold recycled mixtures with emulsified asphalt was evaluated with the resilient modulus (M_R) determined in accordance with ASTM D7369. Given the nondestructive nature of the test, the M_R measurements were conducted on HWTT specimens before performing that destructive rutting/moisture susceptibility test. Therefore, the M_R measurements per emulsified cold recycled mixture type were conducted on three specimens 6 in. (152.4 mm) diameter by 2.5 in. (63.5 mm) height. As with the hot recycled asphalt mixtures, a Poisson's ratio of 0.35 was selected to calculate M_R based on the test temperature (i.e., 77°F [25°C]). After conditioning, a repetitive haversine compressive load pulse was applied in the vertical diametral plane of the specimens and the horizontal deformation was registered through a set of two LVDTs aligned along the diametral plane.

Figure 54 present the average and standard deviation of the M_R measurements for each emulsified cold recycled mixture. The results show that the mixtures fabricated with RAP contents of 60%, regardless of the RAP and virgin aggregate type, developed not only the greatest but quite similar magnitudes of stiffness of around 650 ksi. Likewise, the mixtures with RAP contents of 80% and 100%, independently again of the RAP and virgin aggregate type, developed similar but lower M_R values of around 480 ksi. A reduction of about 26% was observed in the M_R values after incrementing the RAP content from 60% to 80% and 100%.

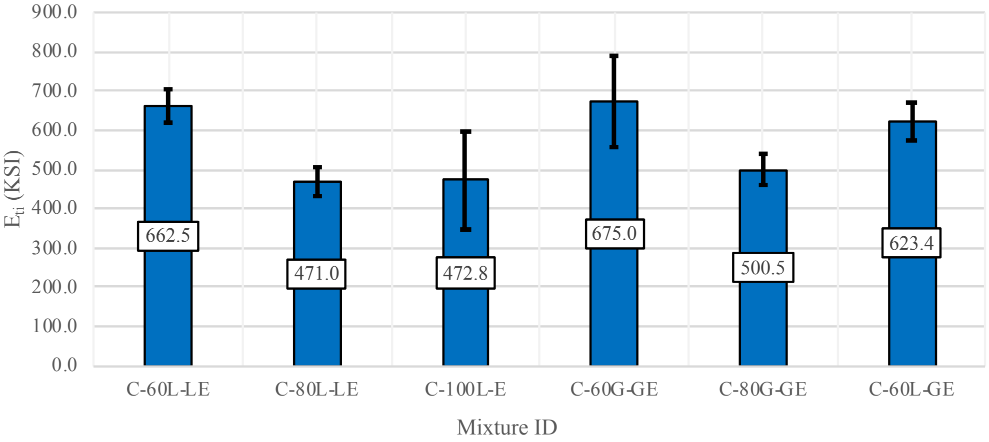


Figure 54. Emulsified Cold Recycled Mixtures Resilient Modulus (M_R) Results

4.3. COLD RECYCLED MIXTURES – FOAMED ASPHALT

The materials employed in the production of the foamed cold recycled asphalt mixtures correspond to the RAP sources (Stockpile 1-09 and Stockpile 1-16) and virgin aggregates types (limestone and granite) listed in Table 5. Additionally, as required per FDOT Standard Specification, Section 334-2.2, a PG 67-22 binder was employed in the design and production of the foamed cold recycled mixtures. The binder rheology was not characterized since it was only used for foaming purposes.

This chapter describes the procedure followed for the design of the cold recycled asphalt mixtures stabilized with foamed asphalt, the specimen preparation and performance results.

4.3.1. Mixture Design

The following sections present the steps followed for the design of cold recycled mixtures with foamed asphalt: (1) optimum foaming water content determination, (2) aggregate blend proportioning, (3) moisture content and curing protocol definition and (4) optimum foamed binder determination.

Two aggregate blends were selected for the design of foamed asphalt cold recycled mixtures (Table 32).

4.3.1.1. *Optimum Foaming Water Content*

In order to achieve proper foaming performance of the PG 67-22 binder, the optimum foaming water content was determined employing expansion ratio (ER) and half-life (H-L) measurements. ER is defined as the ratio between the volume of a specific mass of fluid before and after foaming, while H-L is the period of time that the same fluid takes to transit from its maximum ER to one-half of that value.

The foaming characteristics of the PG 67-22 binder (i.e., ER and H-L) were determined following a novel methodology developed in NCHRP project 09-53 (Newcomb et al. 2015), in which non-contact measurements of the foamed binder height by means of a laser sensor replace the traditional dipstick method. This approach removes the subjectivity associated with the conventional method, because measurements done with the dipstick are generally highly dependent on the visual judgement of the operator.

The laser sensor, known as a laser distance meter (LDM), was setup on a tripod above a standard one-gallon can (Figure 55a) where a sample of foamed binder was dispensed by the Wirtgen WLB 10S employing various foaming water contents. The LDM measured the height of the foamed binder surface by reflecting a laser beam over a very small circular spot (Figure 55b) at a frequency of 1 Hz. Knowing the mass of the dispensed binder sample and the container size, the volume of the sample before foaming can be calculated and the LDM recorded data converted into ER values. An exponential equation was then fitted to the ER versus time data in order to calculate the H-L parameter.


(a) (b)

Figure 55. Foamed Binder Measurements; (a) LDM Equipment Setup and (b) LDM Point Measurement

Figure 56 presents the ER and H-L results of four foaming water contents at a foaming temperature of 320ºF (160ºC). The minimum ER and H-L limits recommended by Wirtgen in their Cold Recycling Technology manual (Wirtgen 2012) are also included and were used to select the optimum foaming water content. Based on both ER and H-L, the results suggested optimum foaming water contents significantly outside the investigated range of selected foaming water contents (Figure 56); therefore, the procedure was repeated at a higher binder foaming temperature of 338ºF (170ºC) (Figure 57).

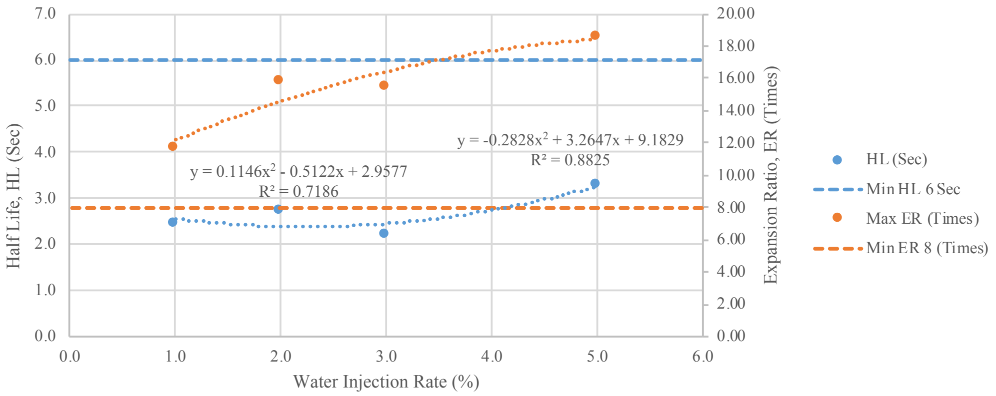


Figure 56. Optimum Foaming Water Content Determination at 320ºF (160ºC)

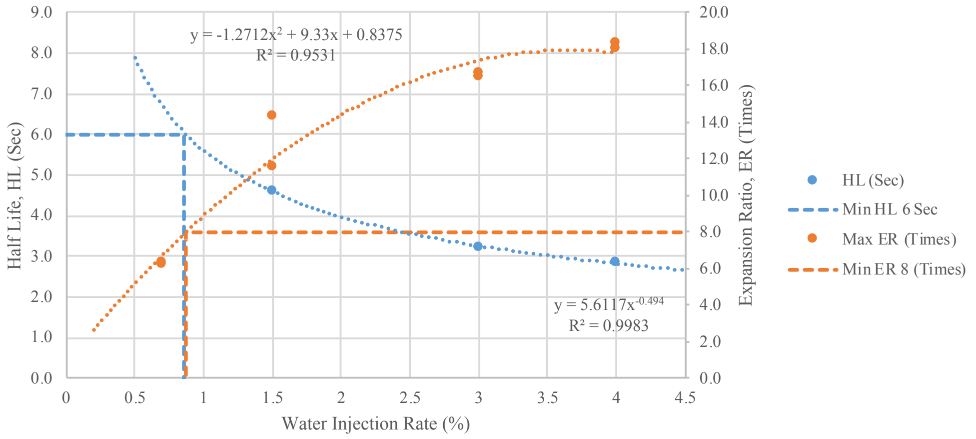


Figure 57. Optimum Foaming Water Content Determination at 338ºF (170ºC)

The results presented in Figure 57 suggest an optimum foaming water content of 0.85% based on both ER and H-L. However, due to practicality and ease of use of the foamer, a water content of 1.0% was selected as optimum. Appendix M presents detailed results of the ER and H-L test.

4.3.1.2. *Material Proportioning*

Two combinations of aggregate type and RAP source (aggregate blends) were selected for the design of the foamed cold recycled mixtures (Table 32).

As defined for the emulsified cold recycled mixtures, the design of foamed cold recycled mixtures was conducted following FDOT base materials specifications. For each aggregate blend, #78 intermediate size stone and W-10 fine screenings were blended with RAP to meet FDOT aggregate gradation requirements established in Standard Specification, Section 234. As for the emulsified cold recycled mixtures, the RAP was considered as a “black rock”. Therefore, the gradations of the RAP before ignition oven (i.e., including the binder coating the RAP material) were employed to meet the specification requirements.

The aggregate blends proportions of #78 stone, W-10 screenings and RAP that met the gradation requirements are shown in Table 32. Figure 58 and Figure 59 show the resulting aggregate gradation curves for each aggregate blend.

Table 32. Foamed Cold Recycled Mixtures Aggregate Blends Proportions

Aggregate's blend	RAP			Virgin Aggregate		
	Source	Aggregate Type	Amount (%)	Type	#78 Stone (%)	W-10 Screenings (%)
ABC-60L-LF	Stockpile 1-09	Limestone	60	Limestone	25	15
ABC-60L-GF	Stockpile 1-09	Limestone	60	Granite	20	20

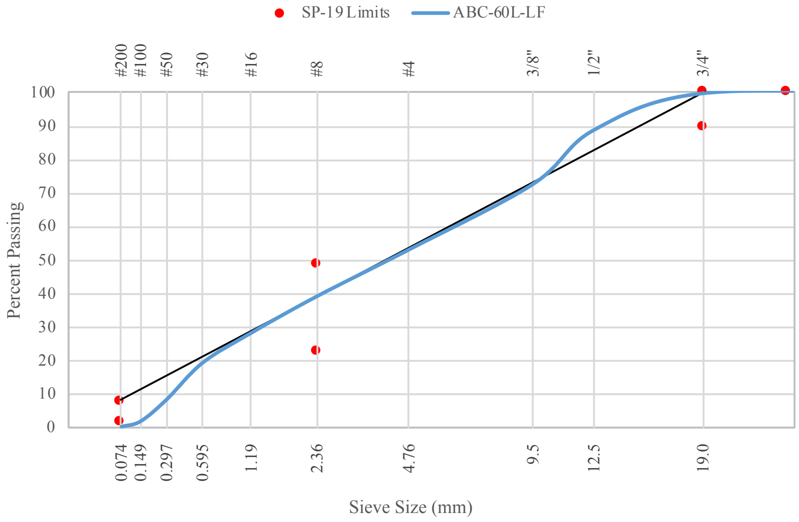


Figure 58. Foamed Cold Recycled Mixtures ABC-60L-LF Aggregate Blend Gradation Curve

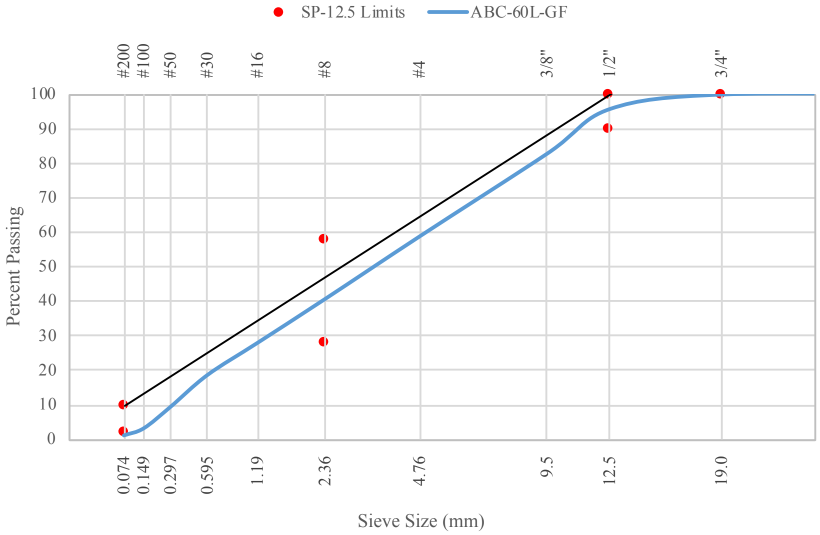


Figure 59. Foamed Cold Recycled Mixtures ABC-60L-GF Aggregate Blend Gradation Curve

The aggregate blend ABC-60L-GE resulted in a NMAS of ½ in. (12.5 mm), whereas the aggregate blend ABC-60L-LE presented a NMAS of ¾ in. (19.0 mm). Therefore, the mixtures produced and requirements verified for the ABC-60L-GE aggregate blend corresponded to B-12.5 and to B-19.0 for the ABC-60L-LE aggregate blend. Detailed aggregate blend calculations and gradation requirements for each NMAS are shown in Appendix C.

4.3.1.3. Moisture Content and Curing Protocol

In order to determine the need for moisture inclusion and curing of foamed cold recycled mixtures, trial mixtures with three binder contents were fabricated employing the ABC-60L-LF aggregate blend listed in Table 32 and moisture contents of 0% and OMC=4%, as determined for the emulsified cold recycled mixtures. Trial specimen were produced and tested for moisture susceptibility as outlined in ARRA standard CR 201.

Four specimens 4 in. (100 mm) diameter by approximately 2.8 in. (70.0 mm) height were fabricated with foamed binder contents of 3% and 5%, employing no water other that the required to foam the binder (i.e., dry aggregates). The mixing and compaction procedures detailed subsequently were followed for the fabrication of the specimens.

Four additional specimens of the same dimensions were fabricated with a foamed binder content of 4.0% but adding 4% water (i.e., moisture content or MC) to the aggregate blend before dispensing the foamed binder. It is important to note that no curing time was provided to these trial specimens.

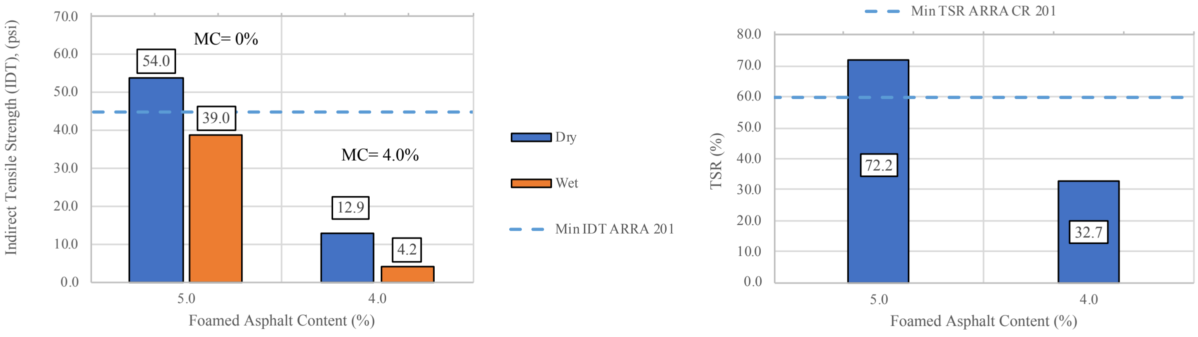
The trial mixtures at 3% and 5% foamed binder content exhibited poor workability when mixing and compacting. Moreover, the trial specimens with 3.0% binder content did not obtain sufficient stability for testing. After compacting, the samples crumbled when ejected from the compaction mold, losing most of their cross section. Figure 60 shows the resulting mixture after dispensing the foamed asphalt and mixing with the aggregate blend. Uncoated aggregate particles and binder lumps covered with fine material were apparent after mixing.

Figure 60. Appearance of Foamed Cold Recycled Mixture with 3% Foamed Binder and No MC after Mixing

The specimens for the rest of the foamed binder contents were placed on a flat surface and allowed to set for at least 3 hours after compaction. After this period, the AV content was determined following FDOT standard test method FM 1-T166 and FM 1-T209. Appendix N presents the bulk specific gravity (G_{sb}) and estimated volumetric properties for each foamed asphalt binder content.

Subsets of two specimens per foamed binder content with the lowest AV content were moisture conditioned in a water bath at room temperature for 24 hours. Two other compacted specimens of the same foamed binder content were tested without conditioning. The IDT strength was determined in accordance with FM 1-T283 for both unconditioned and moisture conditioned specimens.

Figure 61 presents the IDT strength and TSR results. The minimum recommended IDT strength and TSR values according to ARRA CR 201 are also displayed.



(a) (b)

Figure 61. Foamed Cold Recycled Mixture Trial Results; (a) IDT Strength and (b) TSR

The results showed that the foamed cold recycled mixtures with a foamed binder content of 4% and no added OMC met the minimum IDT strength requirement. However, it was difficult to mix and compact these specimens because the mixture was not workable. When an MC of 4% was added to the foamed cold recycled mixtures with 4% foamed binder content, workability improved significantly, but the IDT strength reduced below the ARRA requirement. In addition, after failing these specimens with added MC, a significant amount of moisture was observed inside the specimens as shown in Figure 62.

Figure 62. Cross Section of Foamed Cold Recycled Mixture Trial Specimen with 4% Foamed Binder Content and 4% MC

Based on these results, it was decided to produce the foamed asphalt cold recycled mixtures employing a MC of 4.0% but curing the compacted specimens for 24-hour at a temperature of 140ºF (60ºC), as done for the emulsified cold recycled mixtures. This option was selected in order to avoid the workability issues during mixing and compaction and to also improve the IDT strength of the compacted specimens.

Table 33 summarizes the foamed cold recycled mixture selected MC and curing protocol for the aggregate blends listed in Table 32.

Table 33. Foamed Cold Recycled Mixtures OMC and Curing Time

Cold Recycled Mixture ID	Aggregate's Blend	OMC	Curing Time @ 140 ºF (60ºC)
C-60L-LF	ABC-60L-LF	4%	24h
C-60L-GF	ABC-60L-GF		

4.3.1.4. Optimum Foamed Binder Content

Mixtures with three foamed binder contents were fabricated for each aggregate blend shown in Table 32 in order to find the optimum foamed binder content that could satisfy the minimum IDT strength requirement specified in ARRA standard CR 201.

Before mixing, the aggregates (#78 and W-10) and RAP were oven dried overnight and for four hours, respectively, at 230ºF (110°C). The materials were allowed to cool down to room temperature and then mixed with 4.0% MC. After no more than 5 minutes after adding the water, the binder at the selected amounts was foamed using the optimum foaming water content in the Wirtgen WLB 10S (Figure 63) and mixed with the aggregate blend employing a mechanical mixer.

Figure 63. Wirtgen WLB 10S Foaming Unit

Once mixing was complete, four specimens 4 in. (100 mm) in diameter by approximately 2.8 in. (70 mm) in height were compacted for each foamed binder content in the SGC to N_{Design} = 30 gyrations as established in ARRA standard CR 201. After compaction, the specimens were cured in an oven at 140ºF (60°C) for 24 hours and then allowed to cool down, for at least 3 hours, on a flat surface.

After this period, the AV content of the specimens was determined following FDOT standard test method FM 1-T166 and FM 1-T209. Appendix N presents the bulk specific gravity (G_{sb}) and estimated volumetric properties for each foamed asphalt binder content.

Two specimens per foamed binder content with the lowest AV contents were moisture conditioned in a water bath at room temperature for 24 hours. Two other specimens prepared at the same foamed binder content were left unconditioned. IDT strength was determined in accordance with FM 1-T283 for both unconditioned and moisture conditioned specimens.

Figure 64 and Figure 65 present the IDT strength of the unconditioned and moisture conditioned specimens. Wirtgen recommends minimum IDT strengths of 32.6 psi (225 kPa) and 14.4 psi (100 kPa) for unconditioned and moisture conditioned specimens, respectively (Wirtgen 2012). However, the IDT strength levels developed by the foamed cold recycled mixtures surpassed these thresholds at foamed binder contents as low as 2%. Therefore, a higher and single threshold of 45 psi as prescribed in ARRA standard CR 201, Table 1 was employed to estimate the optimum foamed binder content.

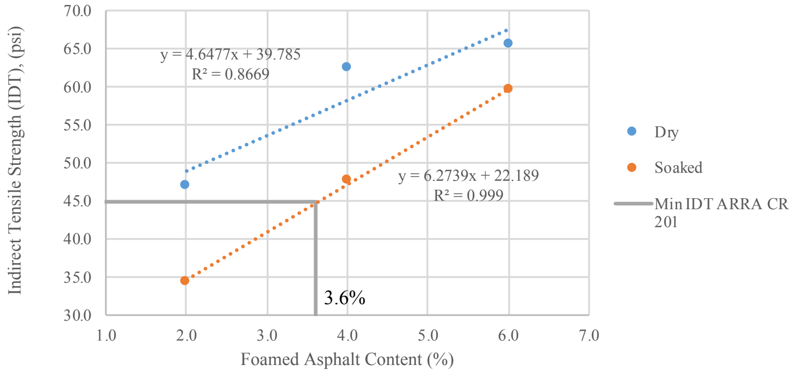


Figure 64. Foamed Cold Recycled Mixture C-60L-LF IDT Strength

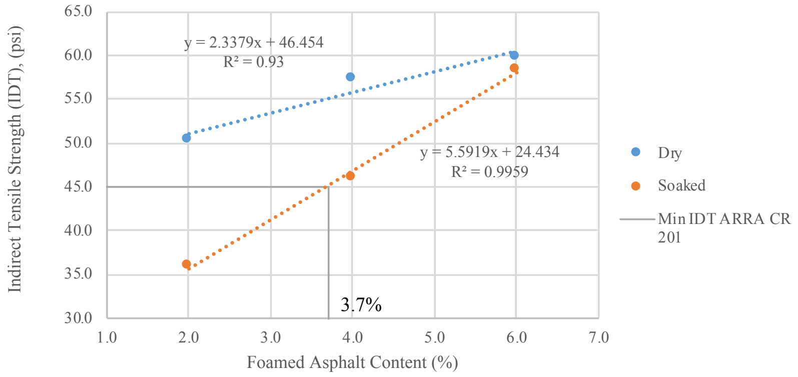


Figure 65. Foamed Cold Recycled Mixture C-60L-GF IDT Strength

Table 34 present the TSR for each mixture type and foamed binder content. A minimum TSR of 60% is established in ARRA standard CR 201, Table 1, for mixtures that incorporate RAP and the moisture conditioned IDT strength exceeds the minimum dry strength/stability requirement of 45 psi.

Table 34. Foamed Cold Recycled Mixtures TSR Results

Foamed Binder Content (%)	Tensile Strength Ratio, TSR (%)	
	C-60L-LF	C-60L-GF
2.0	73	72
4.0	76	80
6.0	91	98

Table 35 presents the optimum foamed binder content for each mixture and the corresponding TSR interpolated from the data presented in Table 34. Based on the resulting TSR values, the mixtures had low moisture susceptibility; therefore, no stabilization by means of addition of lime was considered necessary.

Table 35. Foamed Cold Recycled Mixtures Optimum Foamed Binder Content

Cold Recycled Mixture	Optimum Foamed Binder Content (%)	TSR (%)
C-60L-LF	3.6	79
C-60L-GF	3.7	81

4.3.2. Specimen Fabrication

Specimens for performance testing of cold recycled mixtures stabilized with foamed asphalt were fabricated for the recycled mixtures listed in Table 36. The proportion of aggregate and RAP determined to meet FDOT Standard Specification, Section 234 was presented in Table 32. The optimum foamed binder content was assigned by pairing the virgin aggregate type of the mixture designs presented in Table 35, with the type of virgin aggregate in each mixture (Table 36). Therefore, limestone virgin aggregate mixtures were assigned an optimum foamed binder content of 3.6%, while granite virgin aggregate mixtures were assigned an optimum foamed binder content of 3.7%.

Table 36. Foamed Cold Recycled Mixtures Material Proportions

Mixture Type	RAP Type and Amount		Virgin Aggregate Type Proportioning				Optimum Foamed Binder content (%)	MC (%)
	Limestone (Stockpile 1-09)	Limestone/Granite (Stockpile 1-16)	Limestone		Granite			
			#78 Stone	W-10 Screenings	#78 Stone	W-10 Screenings		
C-60L-LF	60.0%	-	25.0%	15.0%	-	-	3.6	4.0
C-80L-LF	80.0%	-	20.0%	0.0%	-	-	3.6	4.0
C-100L-F	100.00%	-	-	-	-	-	3.6	4.0
C-60L-GF	60.0%	-	-	-	20.0%	20.0%	3.7	4.0

Table 37 presents the list of the tests employed in the performance evaluation of recycled mixtures with foamed asphalt. Specimen characteristics and quantities is also provided. A total of 50 foamed cold recycled mixture specimens were fabricated.

Table 37. Foamed Cold Recycled Mixtures Specimen Characteristics and Quantities

Mixture Property	Test	Standard	Diameter, in. (mm)	Compaction Criteria	Number of Samples per Mixture Type	Total Number of Samples
Moisture Susceptibility	Modified Lottman, Indirect Tension Test (IDT)	FM 1-T283	6 (152.4)	N_{design}=30 gyrations	6	24
Rutting & Moisture Susceptibility	Hamburg Wheel Track Test (HWTT)	AASHTO T 324	6 (152.4)	Height: 2.5 in. (63.5 mm)	4	14
Stiffness	Resilient Modulus (M_R)	ASTM D7369	6 (152.4)	Height: 2.5 in. (63.5 mm)	*	*
Durability	Cantabro	AASHTO TP 108	6 (152.4)	Height: 4.5 in (115.0 mm)	3	12

*The M_R test was conducted on three of the four specimens fabricated for rutting and moisture susceptibility evaluation.

Before mixing, the aggregates (#78 and W-10) and RAP were dried overnight and for four hours, respectively, in an oven at 230ºF (110°C). The materials were allowed to cool down to room temperature and then mixed with the 4.0% OMC employing the mixing chamber of the Wirtgen WLB 10S foaming unit. No more than 2 minutes after and while mixing, the OFAC was dispensed and mixed with the aggregate blend for one minute using the Wirtgen WLB 10S configured to foamed the asphalt at the optimum foaming water content.

Once mixing was complete, the specimens defined in Table 37 were compacted in the SGC to the compaction criteria specified for each specimen type as shown (Table 37).

After compacting, the specimens were placed to cure for 24 hours in a forced draft oven at 140ºF (60ºC). Next, the specimens were taken out of the oven and placed on a flat surface to cool down for at least 24 hours before testing. After this period, the mass of the specimen in air, the mass of the specimen soaked in water and the saturated surface dry (SSD) mass of each specimen was determined as required by Florida's standard test method FM 1-T166. Appendix N presents the bulk specific gravity (G_{sb}) and estimated air voids content for each specimen and mixture defined in Table 37.

One additional sample was fabricated for each cold recycled mixture with foamed asphalt displayed in Table 29 and allowed to cool down at ambient temperature in loose condition. The maximum specific gravity of these mixtures (G_{mm}) was estimated following Florida's standard test method FM 1-T209. The results are also listed in Appendix N.

4.3.3. Performance Results

4.3.3.1. Moisture Susceptibility

The moisture susceptibility of the foamed cold recycled mixtures was evaluated by means of the modified Lottman test as outlined in FDOT test method FM 1-T283.

Due to limitations of the SGC equipment to achieve a compaction height below 1.96 in. (50 mm), three samples 6 in. (152.4 mm) in diameter per foamed cold recycled mixture type were compacted in the SGC using N_{Design} = 30 gyrations as established in ARRA standard CR 201 and cut in half in order to produce six specimens 1.5 in. (38.1 mm) thick.

The six replicate specimens were divided into two subsets of three specimens each according to their AV content. As for the emulsified cold recycled mixtures, the subset with the lowest AV content was moisture conditioned using vacuum saturation plus a 24-hour water bath at room temperature. Ten Hg-in. of partial pressure were applied to each specimen of the subset to achieve vacuum saturation.

After this period, the vacuum was removed and the specimens left submerged for 5 minutes. The other subset of specimens was stored at room temperature throughout the time required to moisture condition the saturated subset. Appendix N presents the volumetric properties and saturation for each specimen.

Both subsets were tested at the same time, after moisture conditioning was completed. IDT strength measurements were conducted at room temperature (77°F [25ºC]) under a monotonic load applied at a rate of 2.0 in./min (50 mm/min), as required by FDOT test method FM 1-T283.

Figure 66 and Figure 67 present the IDT strength and TSR results for the foamed cold recycled mixtures. The minimum IDT strength and TSR requirements according to ARRA standard CR 201 are also displayed.

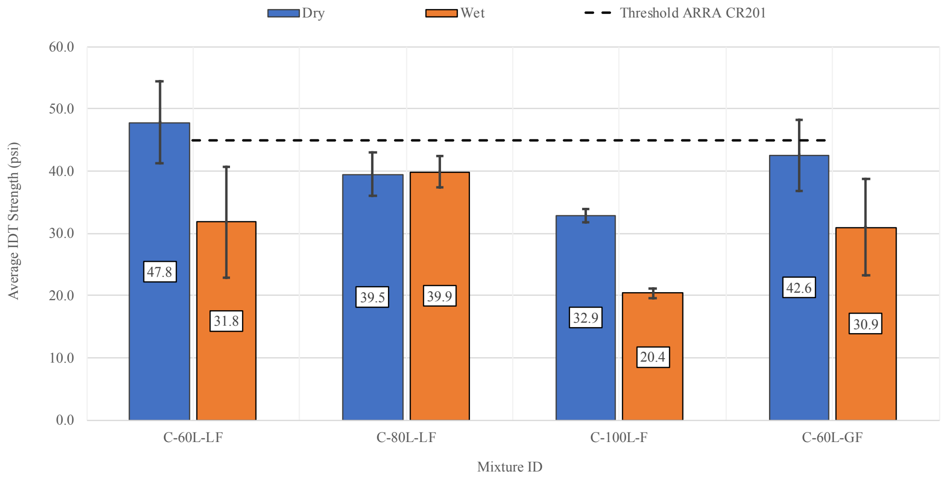


Figure 66. Indirect Tensile Strength of Foamed cold recycled mixtures

According to Figure 66 the foamed cold recycled mixtures exhibit unconditioned IDT strengths that do not meet the minimum ARRA requirement. The mixtures with limestone RAP contents of 60 and limestone virgin aggregate is the only one that developed IDT strengths above the threshold. Regardless of the RAP content and virgin aggregate type, no mixture met the minimum IDT strength requirement after moisture conditioning.

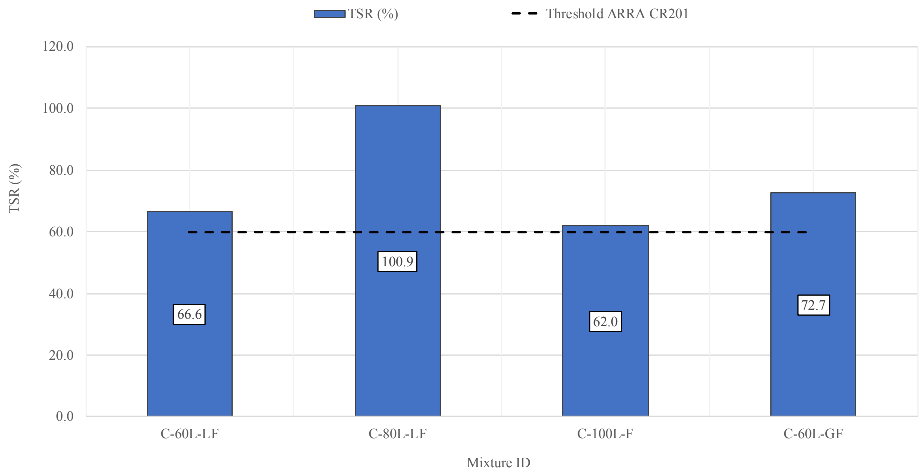


Figure 67. Tensile Strength Ratio of Foamed cold recycled mixtures

From Figure 67, the TSR presented acceptable performance for all the mixtures fabricated with foamed asphalt. However, the mixture with limestone RAP content of 100% barely met the TSR requirements due to a considerable reduction of the tensile strength after moisture conditioning. The results of the mixtures fabricated with limestone virgin aggregate seemed to have a better TSR at a RAP content of 80%. Moreover, the IDT strengths and TSR results were similar for mixtures with 60% limestone RAP regardless of the virgin aggregate type.

4.3.3.2. <u>Rutting & Moisture Susceptibility</u>

Rutting and moisture susceptibility of the foamed cold recycled mixtures was evaluated with the HWTT in accordance with AASHTO T324: *Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA).* The SIP and rut depth at a certain number of load cycles were determined for each mixture type in order to evaluate moisture susceptibility and rutting resistance, respectively. Two test replicates were concurrently conducted per mixture type employing both wheels of the HWTT equipment (i.e., left and right).

Figure 68 and Figure 69 present the SIP obtained on each wheel and the average rut depth versus load cycles, respectively. According to the results in Figure 68, the determination of the SIP parameter was not possible for two of the foamed cold recycled mixtures in either one or both wheels, which indicates that the test specimens did not exhibit stripping throughout the test.

From Figure 68, increasing the limestone RAP content of mixtures fabricated with limestone virgin aggregate did not impact the moisture susceptibility, since both mixtures C-60L-LF and C-80L-LF exhibited approximately the same SIP. With regard to mixture C-60L-GF, the relative lower SIP value (1,500 cycles) indicates that moisture susceptibility is also likely an issue.

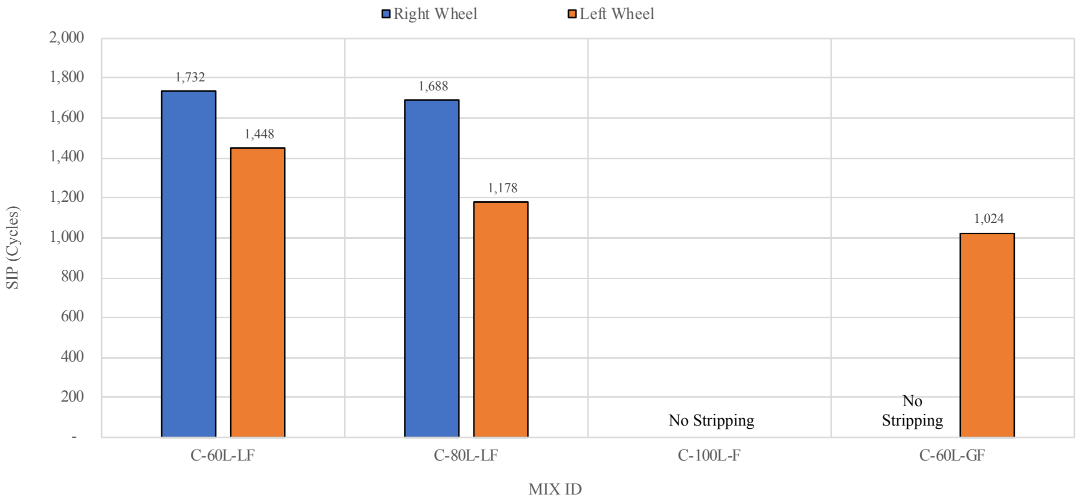


Figure 68. Foamed Cold Recycled Mixtures Stripping Inflexion Point (SIP)

Figure 69 presents the average rut depth of each cold recycled mixture with foamed asphalt. All mixtures experienced accelerated rutting at early load cycles. The assigned rut depth failure criteria of ½ in. (12.5 mm) was reached in less than 2,500 load cycles by all mixtures except C-80L-LF, which reached failure in less than 5,000 cycles.

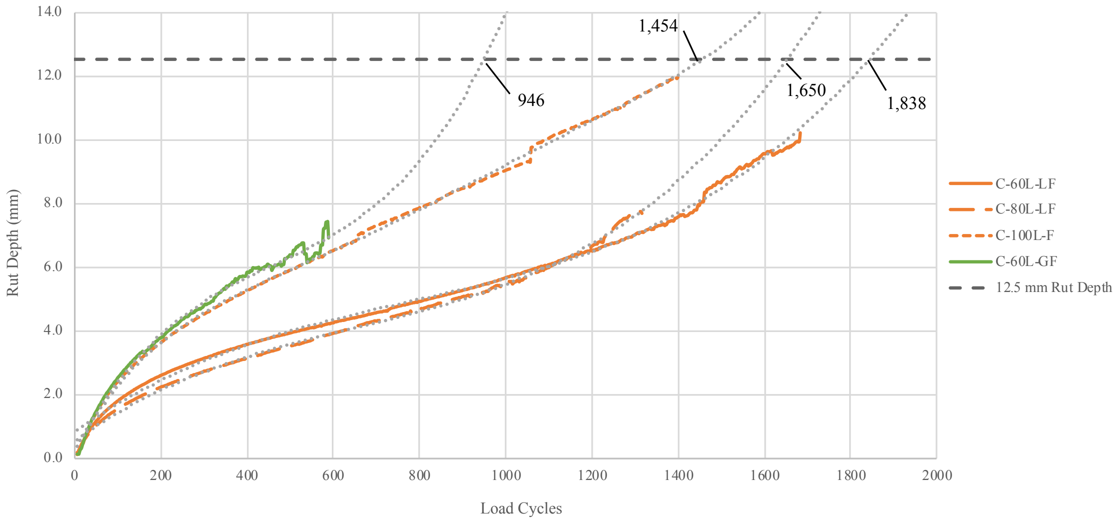


Figure 69. Foamed Cold Recycled Mixtures Ruth Depth vs. Load Cycles

Figure 70 presents the rutting resistance parameter ($\Delta\varepsilon^{vp}{}_{SN}$) values for each foamed cold recycled mixture. Mixtures with limestone virgin aggregate have considerably similar rutting resistance, considering that the average of the mixtures ranges from 59.4 to 64.3. However, replacing the limestone virgin aggregate with granite virgin aggregate in mixtures with 60% limestone RAP content seem to increase the rutting resistance parameter (i.e., mixtures are more prone to rutting) by about 40%.

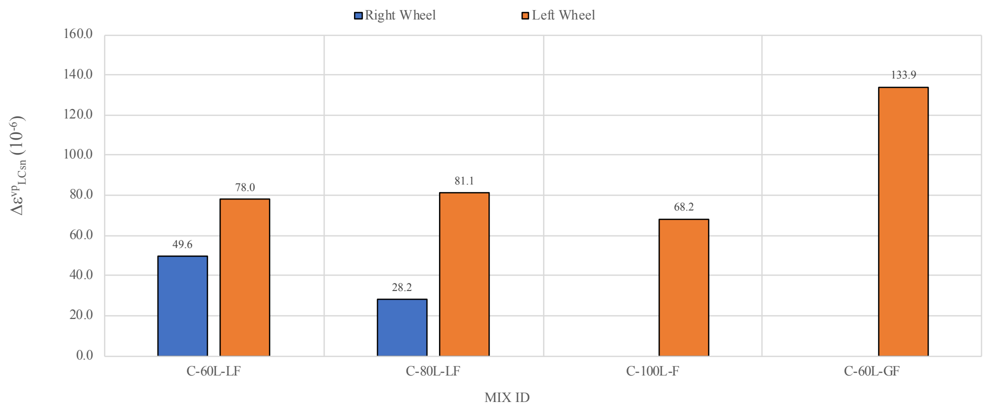


Figure 70. Foamed Cold Recycled Mixtures Rutting Resistance Parameter ($\Delta\ \varepsilon^{vp}_{SN}$)

4.3.3.3. Durability

The durability of the foamed cold recycled mixtures was evaluated using the Cantabro abrasion loss test in accordance with AASHTO TP 108: *Standard Method of Test for Abrasion Loss of Asphalt Mixture Specimens.* Figure 71 presents the average mass loss of compacted specimens after conducting the test. Three replicates per recycled mixture type were tested.

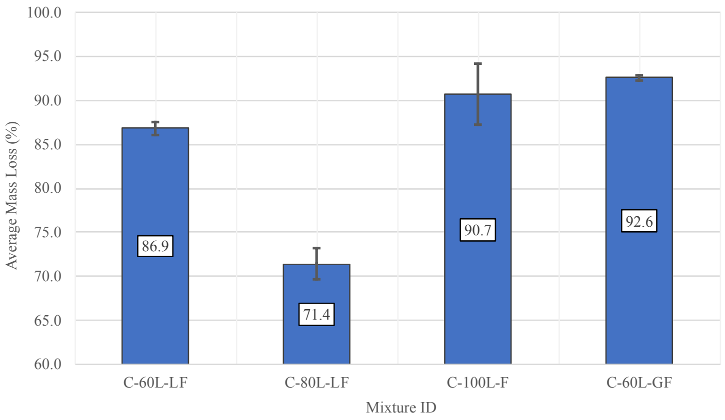


Figure 71. Mass Loss of Foamed cold recycled mixtures after Cantabro Test

All foamed cold recycled mixtures exhibited poor durability, with abrasion mass losses over the recommended maximum of 20% ranging from 71% to 92%. Regardless of the RAP content and virgin aggregate type, the foamed cold recycled mixtures developed considerably high mass loss after testing, which indicates poor cementation between aggregate particles and hence low abrasion resistance. Figure 72 illustrates how the test specimens looked before and after conducting the Cantabro abrasion loss test.

(a) (b)

Figure 72. Specimens Before and After Cantabro Abrasion Loss Test; (a) Mixture C-80L-LF and (b) Mixture C-60L-GF

4.3.3.4. *Stiffness*

The stiffness of the foamed cold recycled mixtures was evaluated by the resilient modulus (M_R) determined in accordance with ASTM D7369: *Standard Test Method for Determining the Resilient Modulus of Bituminous Mixtures by Indirect Tension Test.* Given the nondestructive nature of the test, the M_R measurements were conducted on HWTT specimens prior to rutting/moisture susceptibility testing. The M_R measurements were conducted on three replicate six in. (152.4 mm) in diameter by 2.5 in. (63.5 mm) in height specimens per mixture type. To calculate the M_R value, a Poisson's ratio of 0.35 was assumed based on the test temperature (i.e., 77°F [25°C]). After conditioning, a repetitive haversine compressive load pulse was applied along the vertical diametral plane of the specimens and the horizontal deformation registered through a set of two LVDTs aligned along the diametral plane.

Figure 73 present the average and standard deviation of the M_R measurements per mixture type. The results show that the maximum stiffness was achieved by the mixture fabricated with 80% limestone RAP content. The M_R seems to exponentially decrease with an increase in the RAP content from 80% to 100%. Moreover, the results seem to indicate a low impact of the virgin aggregate type on the foamed cold recycled mixture stiffness. A reduction of about 25% was observed in the M_R value of the C-60L-LF mixture when granite was used instead of limestone as virgin aggregate (C-60L-LG).

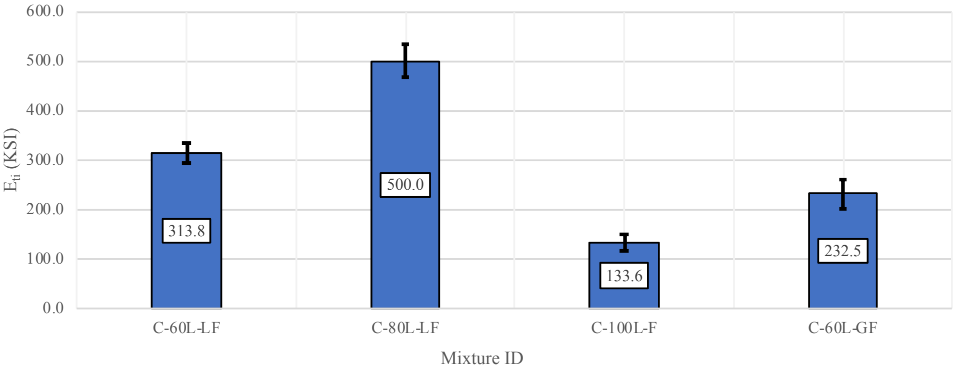


Figure 73. Foamed Cold Recycled Mixture Resilient Modulus (M_R) Results

5. ANALYSIS AND COMPARISON OF RESULTS

The following sections present further discussion regarding the laboratory results obtained for every pavement distress evaluated and performance comparison between the different recycling methodologies.

5.1. MOISTURE SUSCEPTIBILITY

Figure 74 and Figure 75 compare the IDT strength and TSR results of the hot, emulsified and foamed recycled mixtures. The results presented in Figure 74 show that the unconditioned IDT strength of all recycled mixtures, with the exception of C-80L-LF, C-100L-F and C-60LG-F, met the minimum requirement. However, most cold recycled mixtures failed to meet the IDT strength requirement after moisture conditioning. A similar trend was observed for the TSR results; with the *exception* of C-80G-GE and C-60L-GE all mixtures demonstrated adequate moisture susceptibility.

The hot recycled mixtures with no recycling agents (H-60L-L and H-60G-G) developed the largest IDT strengths for both granite and limestone virgin aggregates. As noted before, the addition of recycling agents to the hot recycled mixtures resulted in a reduction of the IDT strength. In the case of mixtures fabricated with limestone/granite RAP and granite virgin aggregate and regardless of the recycling agent type (i.e. organic or petroleum-based), the IDT strength reduction lead to strengths barely 30% greater than emulsified cold recycled mixtures.

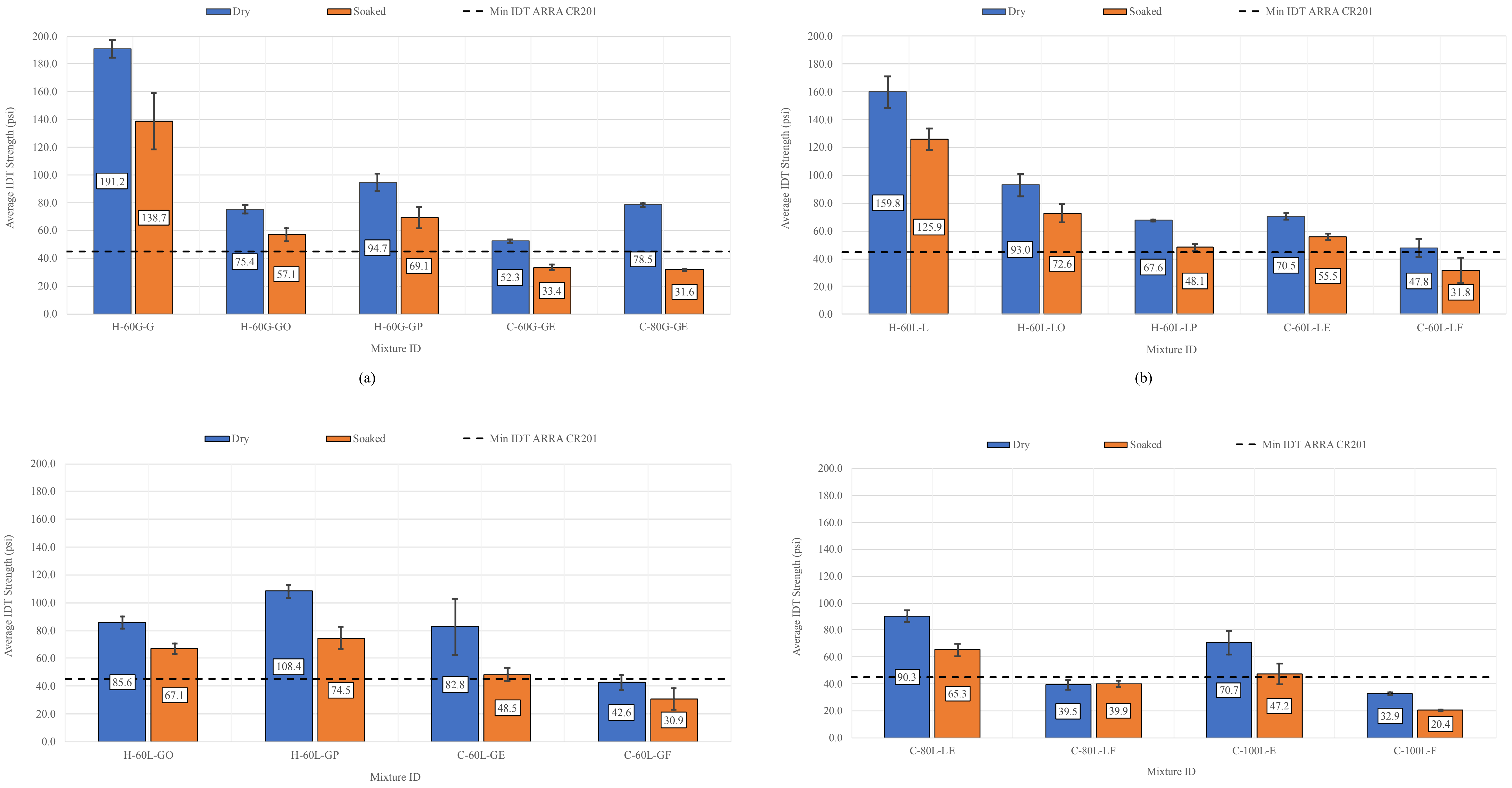
For mixtures fabricated with limestone RAP and limestone virgin aggregate, the IDT strength reduction experienced when adding petroleum-based recycling agents was severe to the point of reaching IDT strength levels equivalent to the emulsified or foamed cold recycled mixtures.

Although the reduction in IDT strength was not as critical when adding organic-based agents, the resulting IDT strength was barely 24% greater than the one obtained for foamed cold recycled mixtures.

Mixtures with limestone RAP and granite virgin aggregate simulate, to a certain extent, the aggregate blend of RAP Stockpile 1-16 (limestone/granite). The largest IDT strengths of these mixtures, at a RAP content of 60%, were developed by those mixtures fabricated through hot recycling methodologies including any: organic or petroleum-based recycling agents (H-60L-GO and H-60L-GP) (Figure 74c). The IDT strengths reached by the emulsified cold recycled mixtures (H-60L-GE) were 23% lower than their HMA counterparts.

Mixtures with higher RAP contents (i.e. 80 and 100%) are more common in cold recycling applications and thus, these RAP contents were evaluated in emulsified and foamed cold recycled mixtures. According to Figure 74d, only the mixtures C-80L-LE and C-100L-E met the IDT strength requirement for unconditioned and moisture conditioned specimens. Mixtures with RAP contents of 100% may achieve the IDT strength requirement by adding hydrated lime or cement.

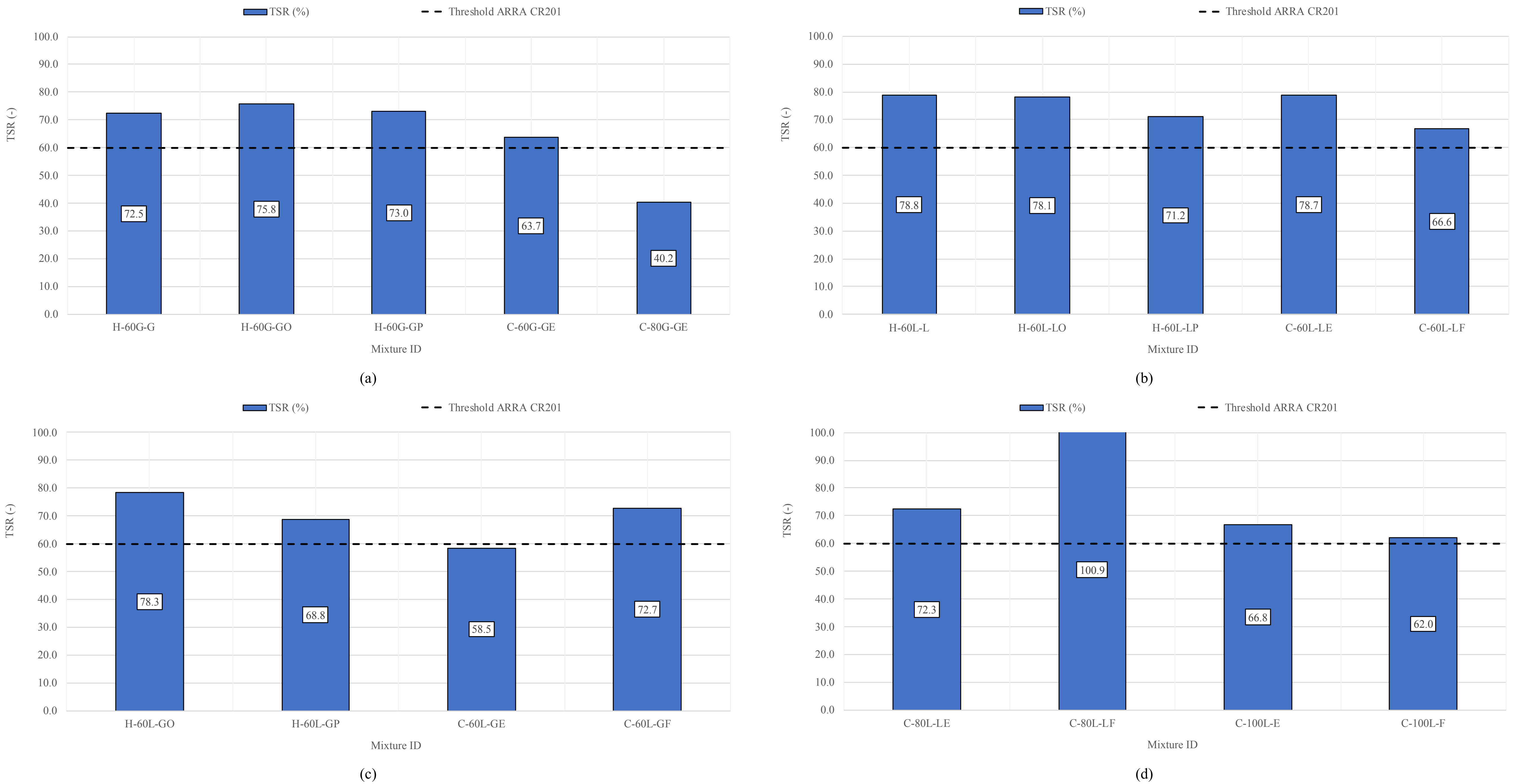
The IDT strength results showed better performance of the hot recycling mixtures that combine granite aggregate (either virgin or present in the RAP) with petroleum-based recycling agent. Moreover, an overall assessment of the IDT strength results shows that cold recycling with foamed asphalt yielded the lowest IDT strengths as compared to the other two recycling methodologies.



(a) (b)

(c) (d)

Figure 74. IDT Strength Comparison



(a) (b)

(c) (d)

Figure 75. TSR Comparison

5.2. RUTTING & MOISTURE SUSCEPTIBILITY

In order to evaluate moisture susceptibility, Figure 76 compares the SIP results of recycled mixtures with similar characteristics produced by means of the three different recycling methodologies.

Based on Figure 76a, some replicates of mixtures with 60% limestone/granite RAP and granite virgin aggregate did exhibit stripping. However, the results show that these mixtures, regardless of the recycling methodology, highlight relatively high moisture susceptibility with low SIP values of around 2,000 cycles. The SIP of the H-60G-GO mixture was the best among all mixtures.

In general, mixtures fabricated with limestone RAP and limestone virgin aggregate through cold recycling with emulsified of foamed asphalt (Figure 76b) had the greatest moisture susceptibility.

The SIP results show the absence of stripping in mixtures fabricated with limestone RAP and granite virgin aggregate (Figure 76c). The moisture susceptibility of these mixtures fabricated through cold recycling is similar to those for recycled mixtures with higher RAP contents of 80% and 100% (Figure 76d). This suggests that the inclusion of RAP when employing cold recycling methodologies has little or no influence on the moisture susceptibility of the mixtures.

As a whole, no recycling methodology presented an improvement with regard to moisture susceptibility or significant difference with respect to the other methodologies.

Figure 77 compares the results of the $\Delta\varepsilon^{vp}{}_{SN}$ parameter to evaluate rutting resistance of recycled mixtures with similar characteristics produced by means of the three different recycling methodologies.

Among the mixtures fabricated with 60% limestone/granite RAP and granite virgin aggregate (Figure 77a), the hot recycled mixtures presented the lowest $\Delta\varepsilon^{vp}{}_{SN}$ values ranging from 2.0 to 21.1, and hence, the greatest rutting resistance. The opposite occurs for mixtures fabricated with 60% limestone RAP and limestone virgin aggregate (Figure 77b), where rutting resistance similar to the hot recycling mixtures can be achieved though emulsified cold recycling as shown by the C-60L-LE mixture.

With regard to the mixtures combining 60% limestone RAP and granite virgin aggregate, poor rutting performance is observed with relative high $\Delta\varepsilon^{vp}{}_{SN}$ values (Figure 77c). Finally, the mixtures with higher RAP contents of 80% and 100% values (Figure 77d) fabricated with emulsified or foamed cold recycled methodologies presented the lowest rutting resistance of all the recycled mixtures with $\Delta\varepsilon^{vp}{}_{SN}$ values up to 81.1.

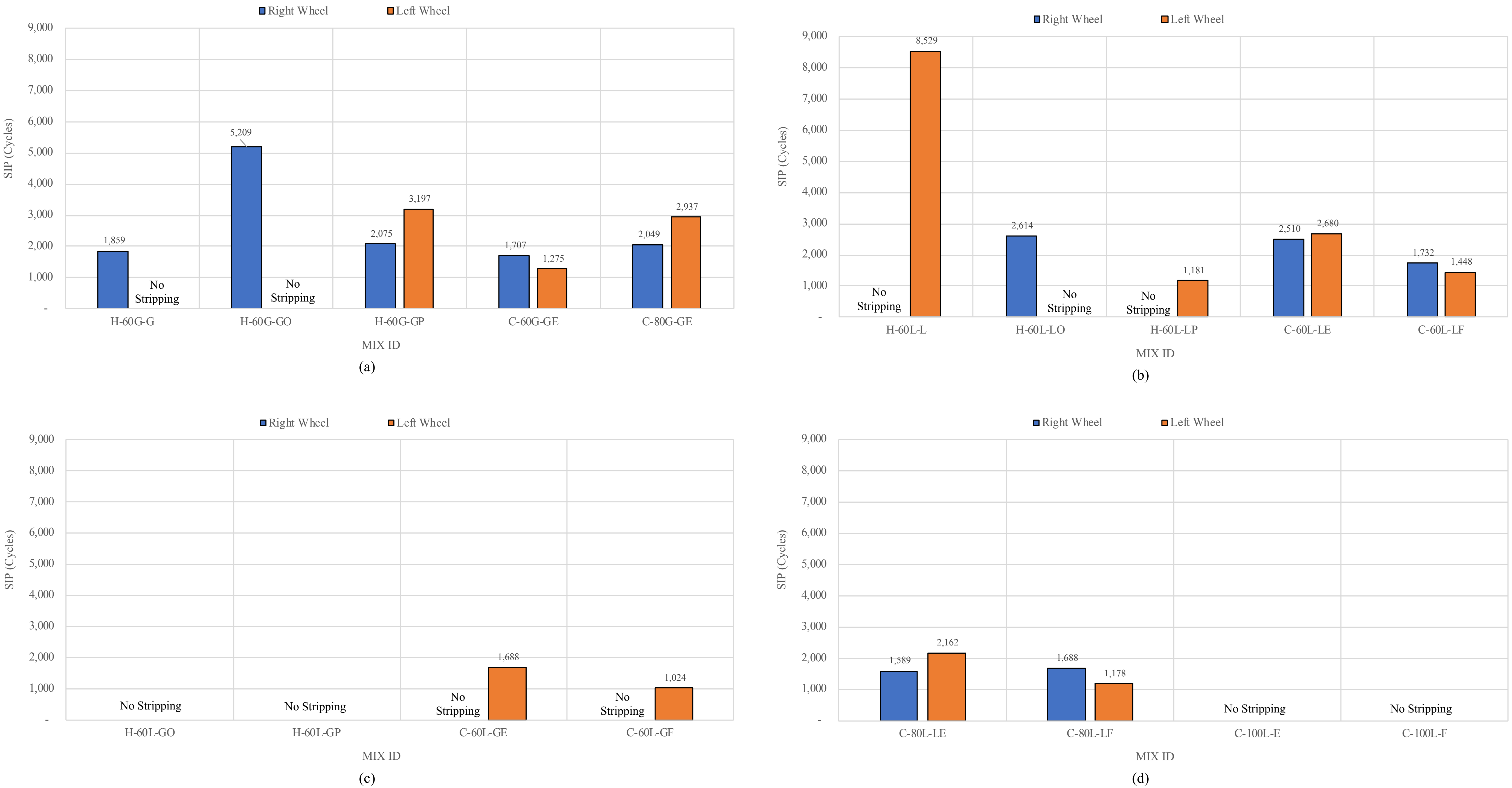
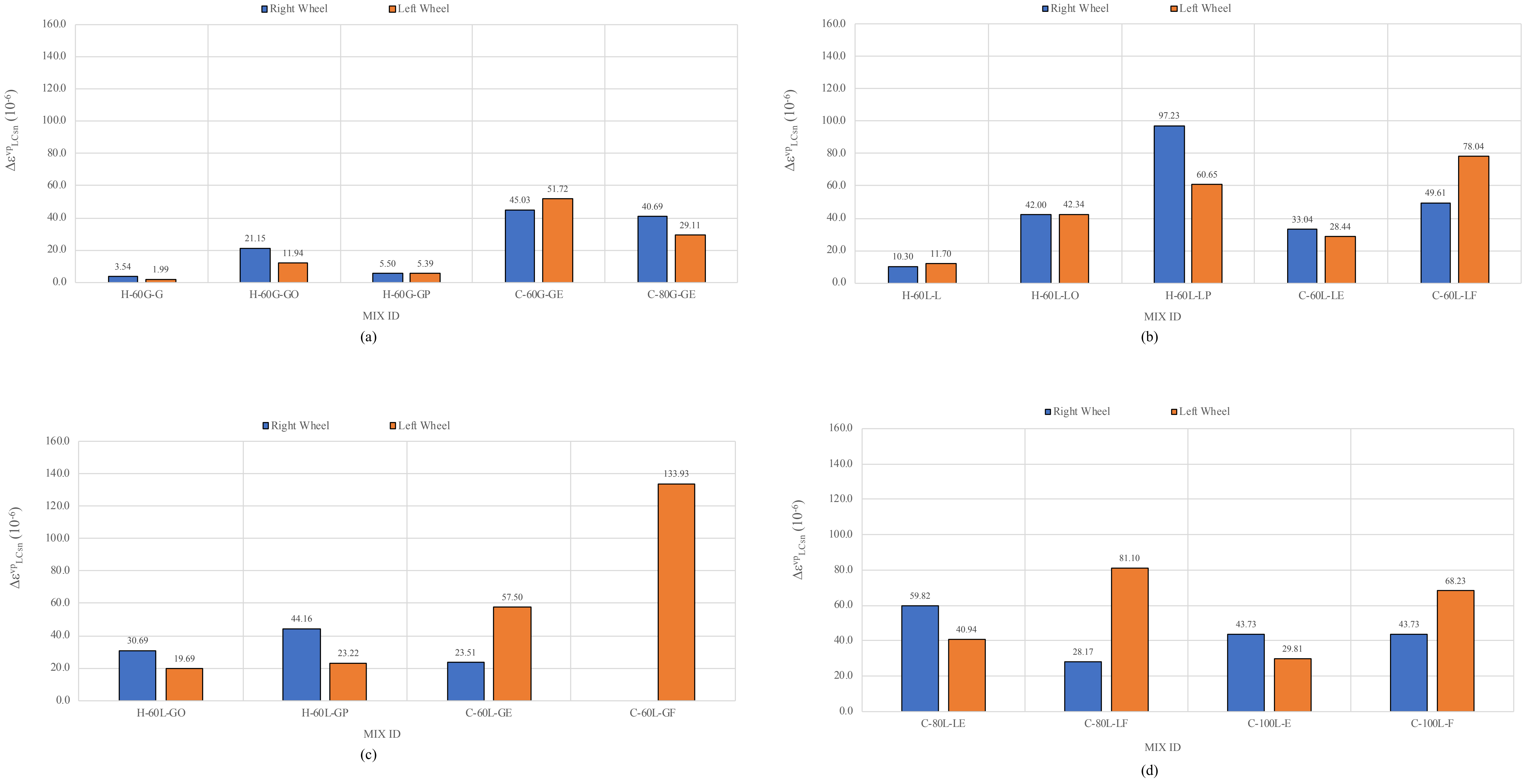


Figure 76. SIP Comparison



(a)

(b)

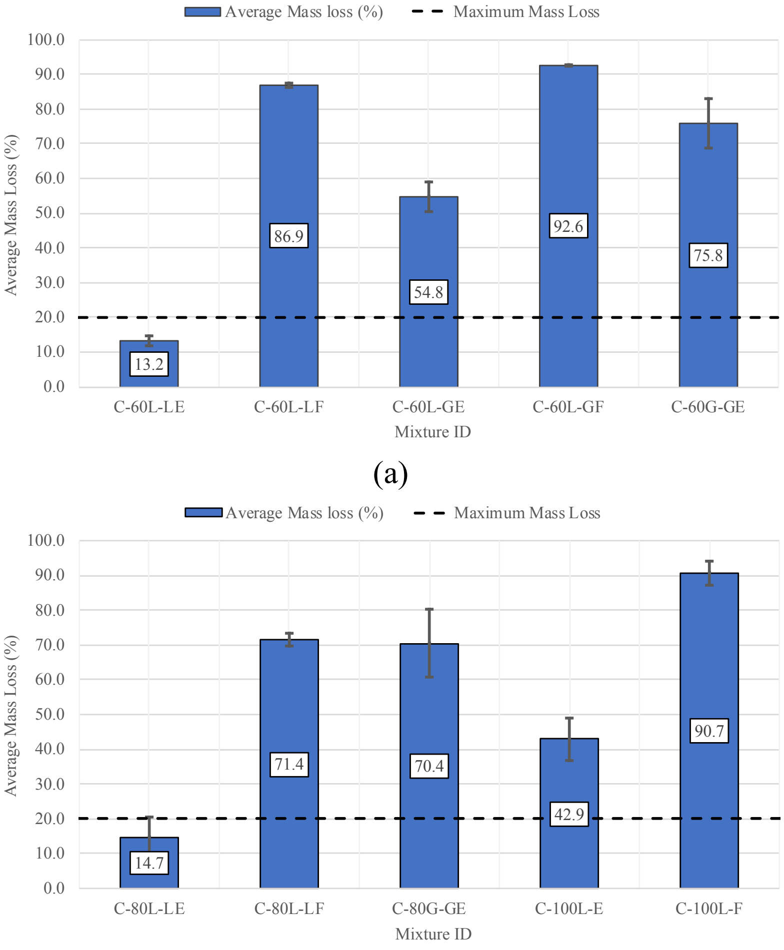
(c)

(d)

Figure 77. Rutting Resistance Parameter ($\Delta\varepsilon^{vp}_{SN}$) Comparison

5.3. DURABILITY

Figure 78 compares the mass loss of cold recycled mixtures subjected to the Cantabro test. Most cold recycled mixtures fabricated with 60% RAP, regardless of the RAP type and recycling methodology, exhibited high mass loss values of up to 92.6% after the test. The C-60L-LE mixture, presented the best raveling performance with 13.2% mass loss (Figure 78a). Likewise, for cold recycled asphalt mixtures with higher RAP contents of 80% and 100%, the C-80L-LE mixture lost 14.7% of its mass (Figure 78b).



(a)

(b)

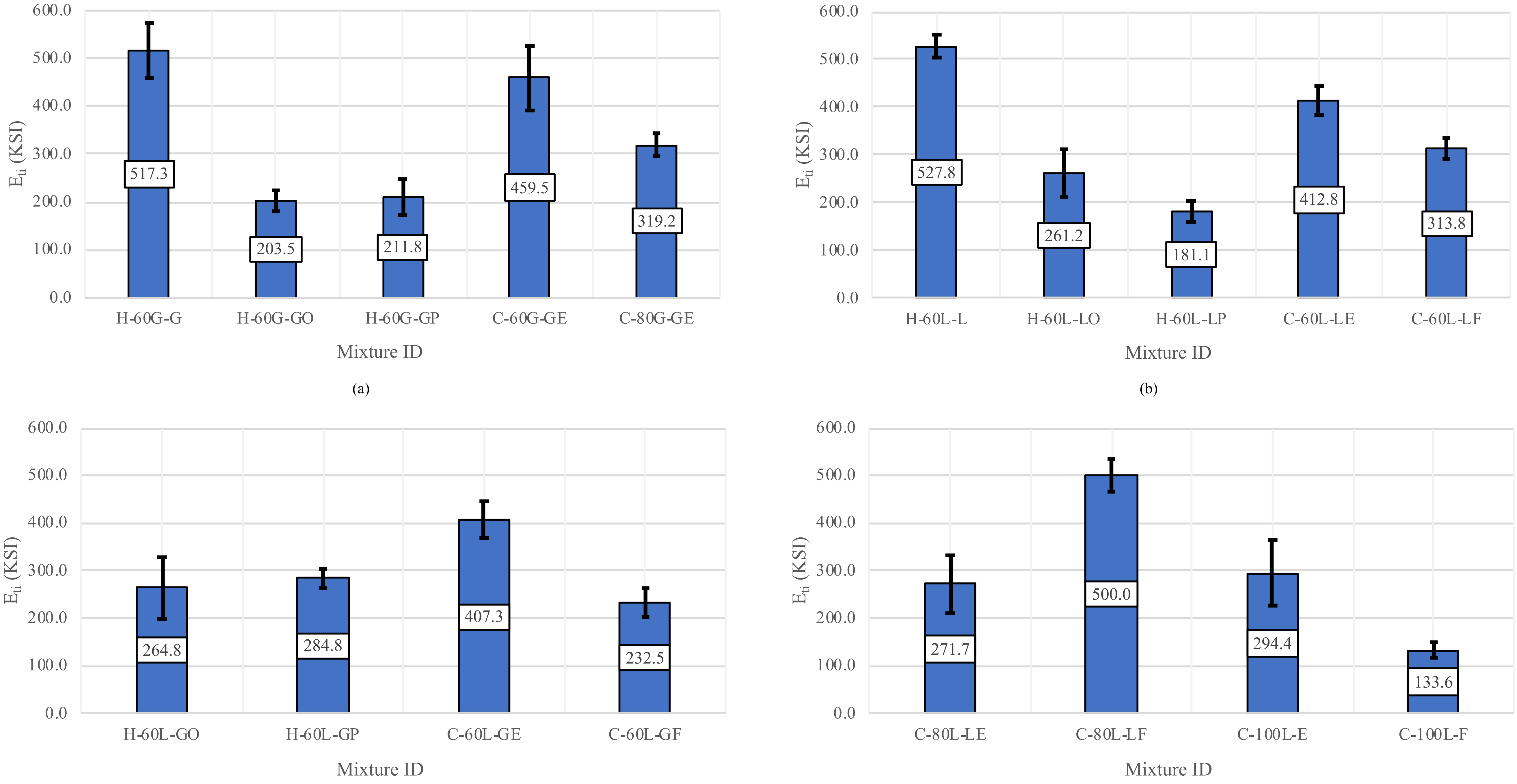
Figure 78. Cantabro Abrasion Mass Loss Comparison

This durability evaluation evidence indicates that the mixtures fabricated with limestone RAP and limestone virgin aggregate though emulsified cold recycling seem to develop a stronger, better quality bonding that provides the mixture with improved durability. Whereas the other types of cold mixtures that were produced with other material combinations or recycling methodologies tended to exhibit poor bonding and hence durability.

5.4. STIFFNESS

Figure 79 compares the M_R results of recycled mixtures with similar characteristics produced by means of the three different recycling methodologies. The results indicate that the stiffness of the hot recycled mixtures with recycling agents of any type tended to be similar regardless of the RAP or virgin aggregate type. The M_R values for the hot recycled mixtures with recycling agents ranged from 181.1 to 284.8 ksi, resulting in the lowest stiffnesses of all the recycled mixtures.

According to Figure 79a through c, the emulsified cold recycled mixtures exhibited in many cases the largest stiffness values, not taking into consideration the mixtures without recycling agents. In the case of RAP contents of 80% (Figure 79d), the foamed recycled mixtures had almost double the stiffness as compared to the emulsified recycled mixtures. For the 100% RAP content mixtures, the trend was opposite, with almost double the stiffness for the emulsified recycled mixture as compared to their foamed counterparts. For the emulsified recycled mixture, the stiffness is practically the same regardless of the RAP content from 80% to 100%.



(a)

(b)

(c)

(d)

Figure 79. Comparison of Resilient Modulus (M_R) Comparison

5.5. INTERACTION PLOTS

In order to assess the global performance of high RAP hot recycled mixtures, Figure 80 to Figure 82 present interaction plots of IDT, TSR and FI results with respect to load cycles until failure in the HWTT. Regions of performance compliance are highlighted and delimited by the performance thresholds for each test. The minimum load cycles before failure due to rutting was defined as 10,000 as recommended by TxDOT Standard Specification Item 358: *Hot In-Place Recycling of Asphalt Concrete Surfaces.*

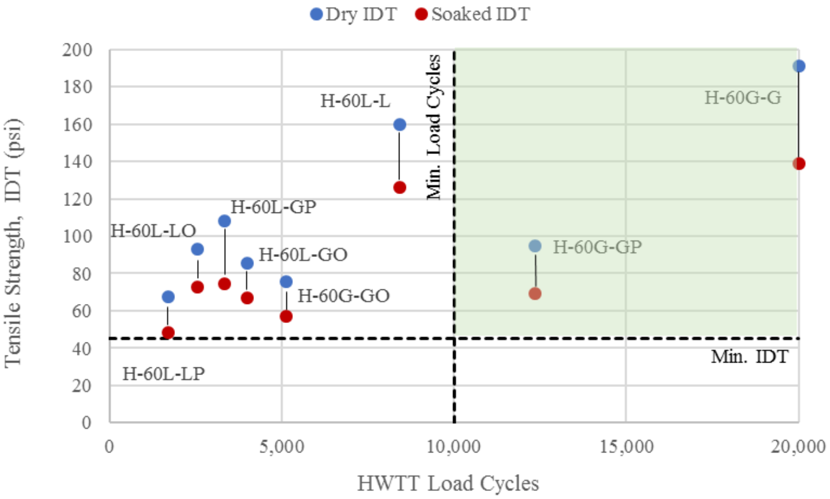


Figure 80. HWTT load Cycles to Failure Vs IDT, Hot Recycling.

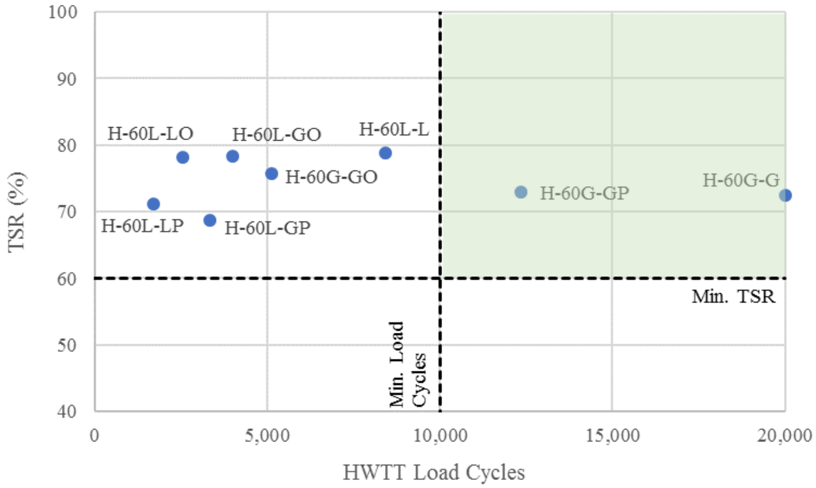


Figure 81. HWTT load Cycles to Failure Vs Tensile Strength Retained (TSR), Hot Recycling.

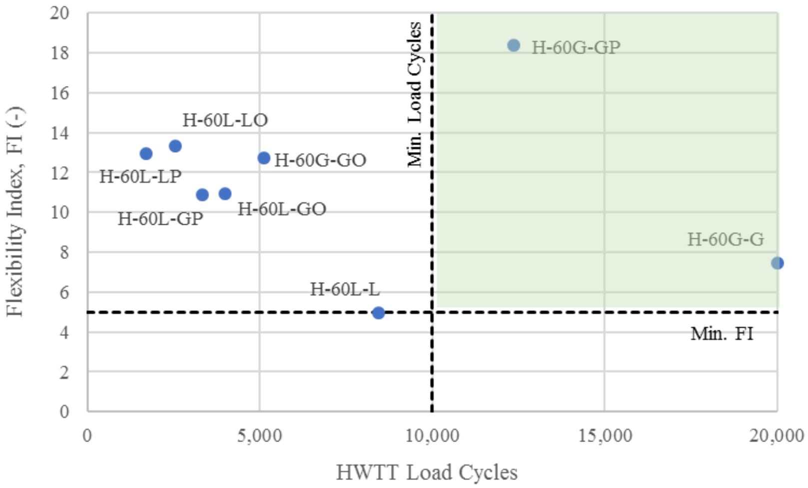


Figure 82. HWTT load Cycles to Failure Vs Flexibility Index Hot Recycling.

According to Figure 80 to Figure 82, resistance to rutting by means of HWTT is controlling the performance of high RAP hot recycled mixtures, since 75% (6 mixtures) of the evaluated mixtures failed to pass rutting requirements. But considering that the performance of most mixtures was adequate in every other test (IDT, TSR and FI), the conventional protocol for rutting evaluation through HWTT is likely too severe for the assessment of high RAP mixtures or the threshold may be too demanding for low volume roads.

However, the mixtures H-60G-G and H-60G-GP met all the requirements including rutting resistance. With the exception of moisture susceptibility, these two mixtures display considerable differences in the performance evaluation. The mixture including no recycling agent (H-60G-G) developed very high IDT strength and rutting resistance but an FI notably close to the minimum threshold, while the mixture including the petroleum-based recycling agent (H-60G-GP) presented better cracking behavior with a higher FI and still very good tensile strength and rutting resistance. These mixtures support the importance of incorporating intermediate temperature cracking tests like SCB in the performance assessment of hot recycled mixtures and at the same time the capability of hot recycling methodologies to produce high RAP hot recycled mixtures with adequate overall performance.

Figure 83 present the interaction plot between IDT results and load cycles until failure in the HWTT for the high RAP cold recycled mixtures stabilized with emulsion. The requirement for load cycles before failure due to rutting was defined to be no less than 5,000 but no more than 15,000 as recommended by TxDOT Special Standard S.S. 3254: *Cold In-Place Recycling of Asphalt Concrete Pavement*.

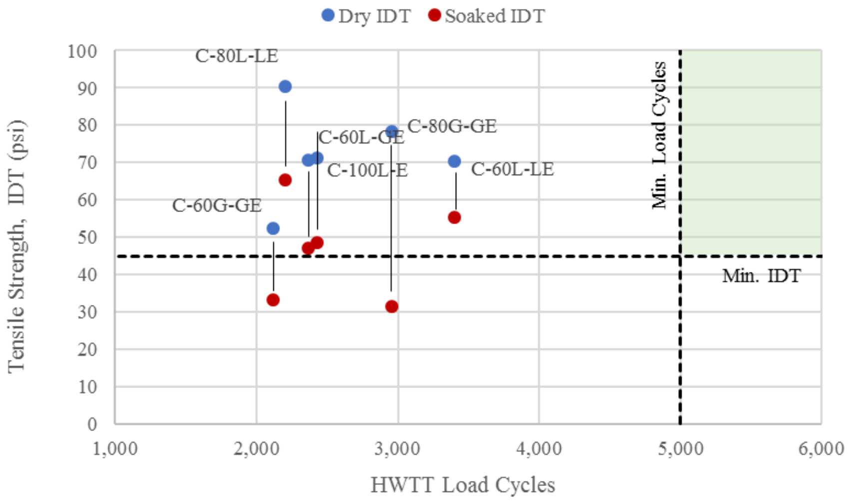


Figure 83. HWTT load Cycles to Failure Vs IDT, Cold Recycling – Emulsion

According to Figure 83, none of the high RAP mixtures with emulsion met the minimum requirement of 5,000 load cycles before failure in the HWTT. The mixture C-60L-LE presented the maximum resistance to rutting with approximately 2/3 of the minimum threshold (i.e. 3,400 cycles). Moreover, most of the mixtures presented a more critical performance developing cycles to failure below one-half of the minimum threshold (i.e. 2,500 cycles). Therefore, the conventional protocol for rutting evaluation through HWTT is likely too severe for the assessment of high RAP cold recycled mixtures.

Considering that HWTT does not facilitate the global performance assessment of cold recycled mixtures with emulsion and the base-material nature of cold recycled mixtures, Figure 84 and Figure 85 present the interaction of IDT and TSR results with respect to Mass Loss in the Cantabro test. Only the mixtures C-60L-LE and C-80L-LE met the performance requirements including tensile strength, moisture susceptibility and durability.

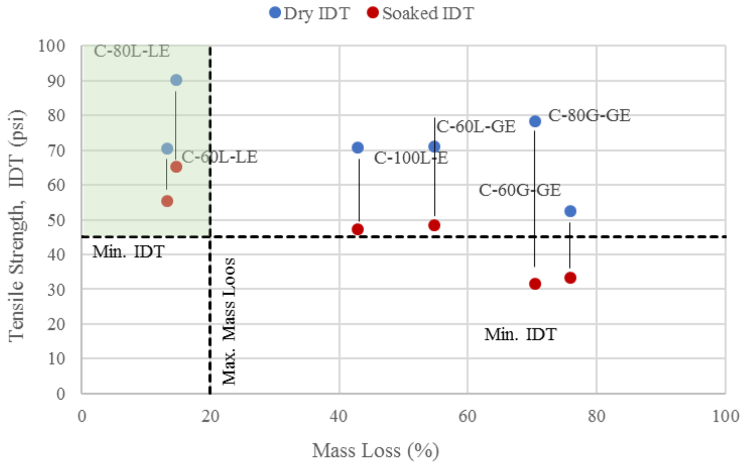


Figure 84. Cantabro Mass Loss Vs IDT, Cold Recycling – Emulsion

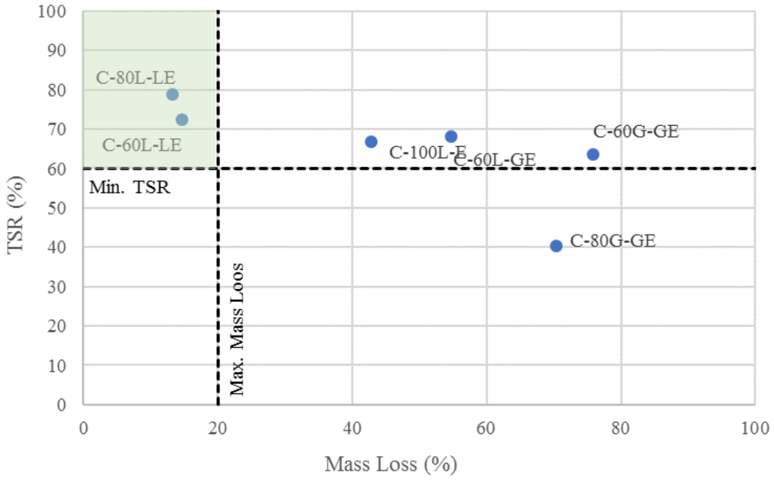


Figure 85. Cantabro Mass Loss Vs Tensile Strength Retained (TSR), Cold Recycling – Emulsion

These two mixtures (C-60L-LE and C-80L-LE) support the necessity of incorporating durability tests such as Cantabro in the performance assessment of cold recycled mixtures, and at the same time the ability of cold recycling methodologies with emulsified asphalt to produce high RAP cold recycled mixtures with adequate overall performance.

6. CONCLUSIONS

In this chapter, a summary of general and more specific observations for each recycling methodology after performance analysis is provided.

6.1. HOT RECYCLED MIXTURES

The high RAP hot recycled mixtures fabricated in this research project were tested in the laboratory to assess their performance in relation to: moisture susceptibility, rutting and cracking resistance and stiffness by indirect tensile strength, modified Lottman, HWTT, Semicircular Bending Beam (SCB) test, and Resilient Modulus (M_r) the results were compared to current thresholds for recycled mixtures and HMA.

Current procedures for the design of recycled mixtures through IDT and TSR performance were proven unable to capture cracking problems that some mixtures meeting requirements might present. Evaluation through of SCB test demonstrated to be effective at detecting recycled mixtures with greater ductility. The incorporation of cracking assessment in the design of high RAP hot recycled mixtures is recommended in addition to the evaluation of tensile strength and moisture susceptibility.

The inclusion of rutting thresholds based on HWTT demonstrated to limit the mixtures meeting performance criteria to a few combinations of virgin aggregate, RAP source and recycling agent type. The conventional protocol for rutting evaluation through HWTT is likely too severe for the assessment of high RAP hot recycled mixtures or the threshold may be too demanding for low volume roads. It is recommended further research regarding rutting performance of high RAP hot recycled mixtures by means of dry tests such as Asphalt Pavement Analyzer (APA), in order to avoid the use of hot water in the process.

Finally, high RAP hot recycled mixtures demonstrated the capability to perform as surface layers for pavements of low volume roads according to requirements of tensile strength and moisture susceptibility as defined by current standards for design of recycled mixtures.

The following is a summary of specific observations and conclusions regarding the performance of high RAP hot recycled mixtures:

- Among the recycling agents evaluated, the organic and petroleum-based recycling agents of B-1 and P-1, respectively, displayed the lowest susceptibility to laboratory aging.
- Workability and CI tests displayed no significant difference between adding the recycling agent to the virgin binder and the alternative of letting the agent "marinate" the RAP before mixing.
- The IDT strength was lower for the hot recycled mixtures after incorporating the recycling agents (both organic and petroleum-based types) in the recycled mixtures. The decrease in strength ranged from 32% to 61% for unconditioned specimens and from 41% to 62% for moisture conditioned specimens. Despite this reduction, all the hot recycled mixtures evaluated had adequate moisture susceptibility performance with respect to the minimum IDT strength after moisture conditioning.
- All TSR results were also above the minimum recommended by ARRA CR201 of 60%, suggesting low moisture susceptibility.
- In most of the cases, the hot recycled mixtures displayed better or similar tensile strength performance as compared to all cold recycled mixtures.

- Most hot recycled mixtures experienced accelerated rutting in the HWTT at early life. The rut depth failure criteria of ½ in. (12.5 mm) was reached in less than 5,000 load cycles by all mixtures except mixture H-60G-GP and H-60G-G.
- High RAP hot recycled mixtures tended to fail in the HWTT without developing stripping. With the exception of mixture H-60G-GP, at least one replicate of all the hot recycled mixtures did not exhibit stripping throughout the test.
- The mixtures with recycling agents improved their FI with respect to those without recycling agents from 46% to 145% for granite virgin aggregate mixtures and around 160% for limestone virgin aggregate mixtures.
- The inclusion of recycling agents helped control the cracking performance of high RAP hot recycled mixtures.
- The reduction in stiffness in the mixtures with recycling agents was in the range of 36% to 57% for the granite virgin aggregate and from 38 to 60% for the limestone virgin aggregate.
- H-60G-G and H-60G-GP were the only two mixtures that met all the performance requirements including tensile strength, moisture susceptibility, cracking and rutting resistance.

6.2. COLD RECYCLED MIXTURES

The high RAP cold recycled mixtures fabricated in this research project were tested in the laboratory to assess their performance in relation to: moisture susceptibility, rutting resistance, durability and stiffness by indirect tensile strength, HWTT, Cantabro abrasion loss test, and resilient modulus (M_r). The results were compared to current thresholds for recycled mixtures.

The following is a summary of specific observations and conclusions regarding cold recycled mixtures stabilized with emulsified and foamed asphalt:

- The OMC obtained from moisture-density curves resulted in elevated water contents that reduced the stability of compacted specimens.
- The moisture conditioning protocol defined by FM 1-T283 that includes freeze/thaw cycle and a 24-hour warm bath at 140°F (60°C) was too aggressive for the cold recycled mixtures. A reduced moisture conditioning protocol consisting of vacuum saturation plus a 24-hour water bath at room temperature is recommended.
- A 24-hour curing period in a forced draft oven at a temperature of 140°F (60°C) provided mass stabilization of compacted specimens.
- C-60L-LE and C-80LE were the only two mixtures that met all the performance requirements including tensile strength, moisture susceptibility and durability.

6.2.1. Emulsified Asphalt

Current procedures for the design of recycled mixtures through IDT and TSR performance were found unable to capture durability problems that some mixtures exhibited. Considering the base-material nature of cold recycled mixtures, durability tests as Cantabro are recommended for better characterization of mixture performance, although durability evaluations might result in a more demanding design process. Further investigation is recommended with regard to the inclusion of hydrated lime to improve the tensile strength and moisture susceptibility performance of the emulsified cold recycled mixtures.

The conventional protocol for rutting evaluation through HWTT is likely too severe for the assessment of high RAP cold recycled mixtures stabilized with emulsion. Assessment of rutting performance through dry tests such as APA is recommended for further investigation.

Finally, the emulsified cold recycled mixtures demonstrated the capability to perform as surface layers for pavements of low volume roads according to requirements of tensile strength and moisture susceptibility as it is defined by current standards for design of cold recycled mixtures

The following is a summary of specific observations and conclusions regarding the performance of high RAP cold recycled mixtures stabilized with emulsified asphalt:

- Considering the high variability of the IDT strength exhibited by the mixture C-60L-GE, the largest IDT strength was achieved by mixtures with 80% RAP content.
- Mixtures fabricated with higher RAP contents developed lower IDT strengths after moisture conditioning
- The tensile strength and moisture susceptibility performance of mixtures including limestone RAP and limestone virgin aggregate was adequate based on minimum requirements.
- The mixtures fabricated with limestone/granite RAP at contents of 80% did not meet the TSR requirement due to considerable reduction of the tensile strength after moisture conditioning.
- All mixtures experienced accelerated rutting at early life. The rut depth failure criteria of ½ in. (12.5 mm) was reached in less than 3,500 load cycles in all cases.
- The mixtures including granite virgin aggregate exhibited low durability with Cantabro mass loss values ranging from 55% up to 76%, possibly indicting poor bonding between the granite aggregate and the emulsified binder.

- The mixtures fabricated with limestone virgin aggregate and limestone RAP contents of 80% and lower (C-60L-LE and C-80L-LE) presented adequate durability performance with Cantabro mass loss values under the 20% recommended threshold.
- The low Cantabro mass loss results presented by the mixtures including limestone virgin aggregate and limestone RAP contents up to 80% suggest better bonding characteristics of the limestone aggregate with the emulsified binder.
- A reduction of about 26% in the mixture stiffness was detected after increasing the RAP content from 60% to 80% or 100%.
- The mixtures' stiffness seemed to reduce and stabilize at a certain value after increasing the RAP content from 60 to 80%.
- In all of the cases, the emulsified cold recycled mixtures displayed better tensile strength performance than all foamed recycled mixtures.
- The emulsified cold recycled mixtures registered in all the cases larger stiffness values than hot recycled mixtures with recycling agents (i.e. organic or petroleum-based).

6.2.2. Foamed Asphalt

The laboratory results showed that none of the foamed cold recycled mixtures did not meet the tensile strength and durability requirements. Therefore, further investigation with regard to the inclusion of Portland cement is recommended to possibly improve the IDT strength and durability of these mixtures.

The following is a summary of specific observation and conclusions regarding the performance of high RAP cold recycled mixtures stabilized with foamed asphalt:

- The addition of moisture to the aggregate blend before mixing with the foamed asphalt provided an important improvement to the workability of the mixture.
- Although good moisture susceptibility was observed in all the foamed cold recycled mixtures, none of them met the minimum IDT strength requirement before and after moisture conditioning with the exception of C-60L-LF, which only passed dry IDT strength.
- One of the mixtures (C-100L-F) did not show evidence of stripping throughout the HWTT test. However, all mixtures experienced accelerated rutting. Rut depth failure criteria of ½ in. (12.5 mm) was reached in less than 2,000 load cycles by all foamed cold recycled mixtures.
- All foamed cold recycled mixtures presented poor durability with considerable high Cantabro mass loss, ranging from 71% to 92%, suggesting poor adhesion between aggregate particles.
- Maximum M_R stiffness was achieved by the mixture C-80L-LF.
- M_R decreased with an increase of RAP content from 80% to 100%.

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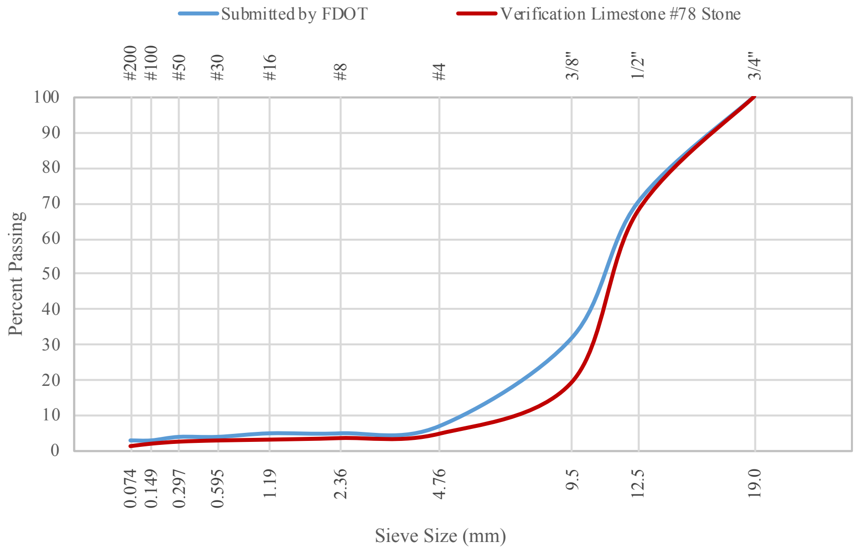
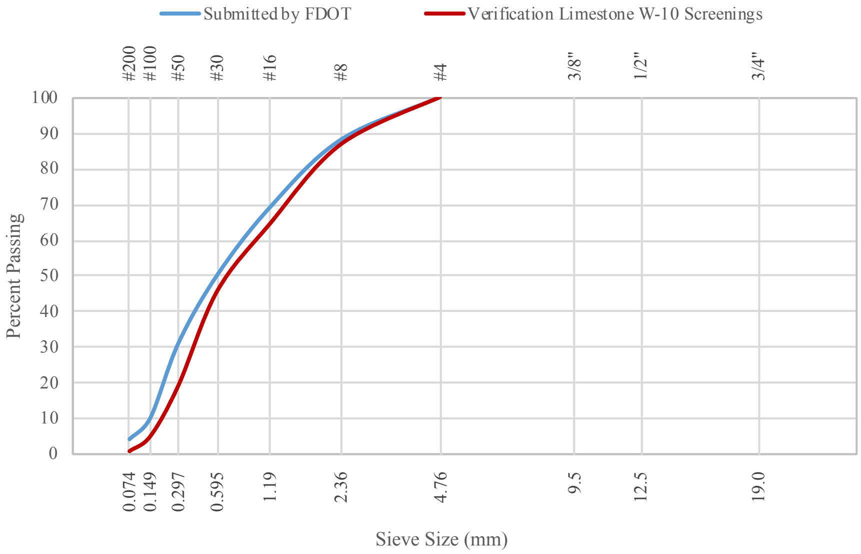
APPENDIX A

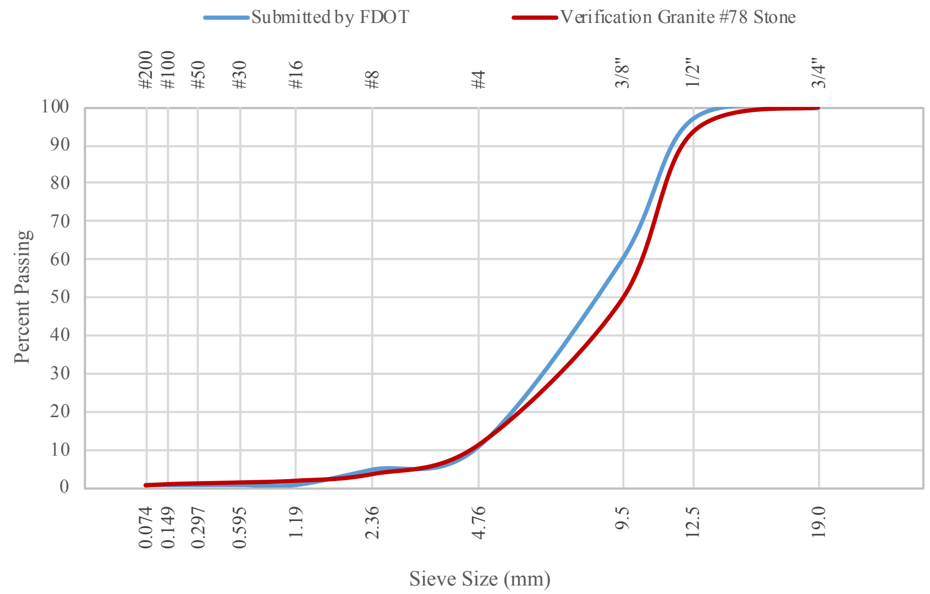
AGGREGATE AND RAP GRADATIONS



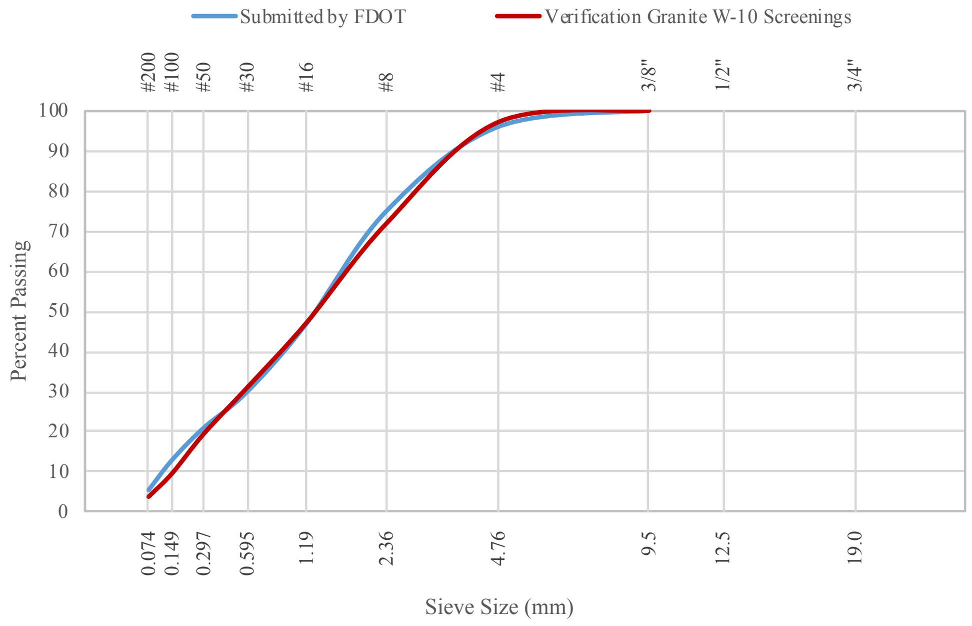
Gradation curve #78 Stone of Limestone

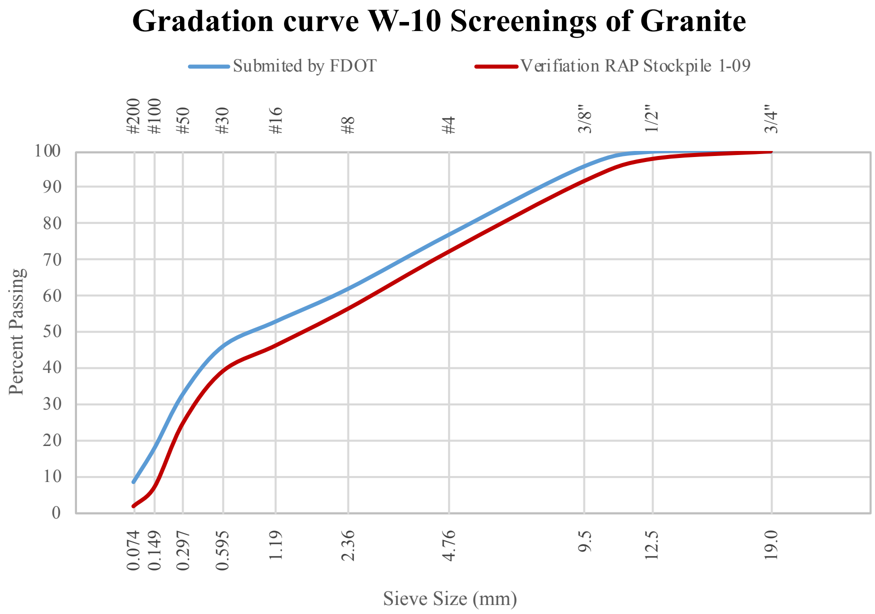


Gradation curve W-10 Screenings of Limestone

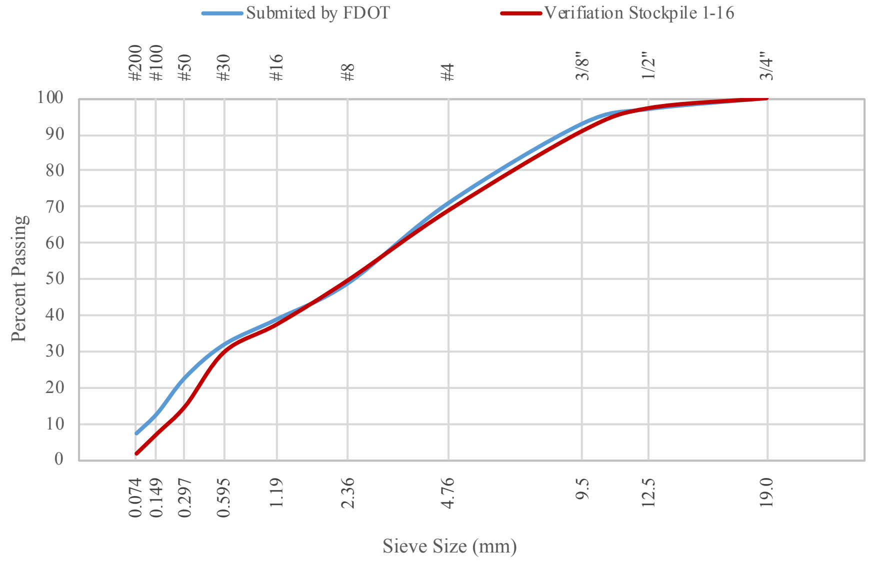


Gradation curve #78 Stone of Granite





Gradation Curve after Ignition Oven RAP Stockpile 1-09 Limestone



Gradation Curve after Ignition Oven RAP Stockpile 1-16 Limestone/Granite

APPENDIX B

AGGREGATE PROPERTIES REPORTS

Specific Gravity # 78 Stone Limestone*

Geographic District	Managing District	Mine	Terminal	Facility Type	Product	Process	Description	Depleted?	Gsb
DISTRICT 6	DISTRICT 6	87339		Mine	C41	1	S1A Stone		2.775
DISTRICT 6	DISTRICT 6	87339	TM 427	Terminal	C41	1	S1A Stone		2.775

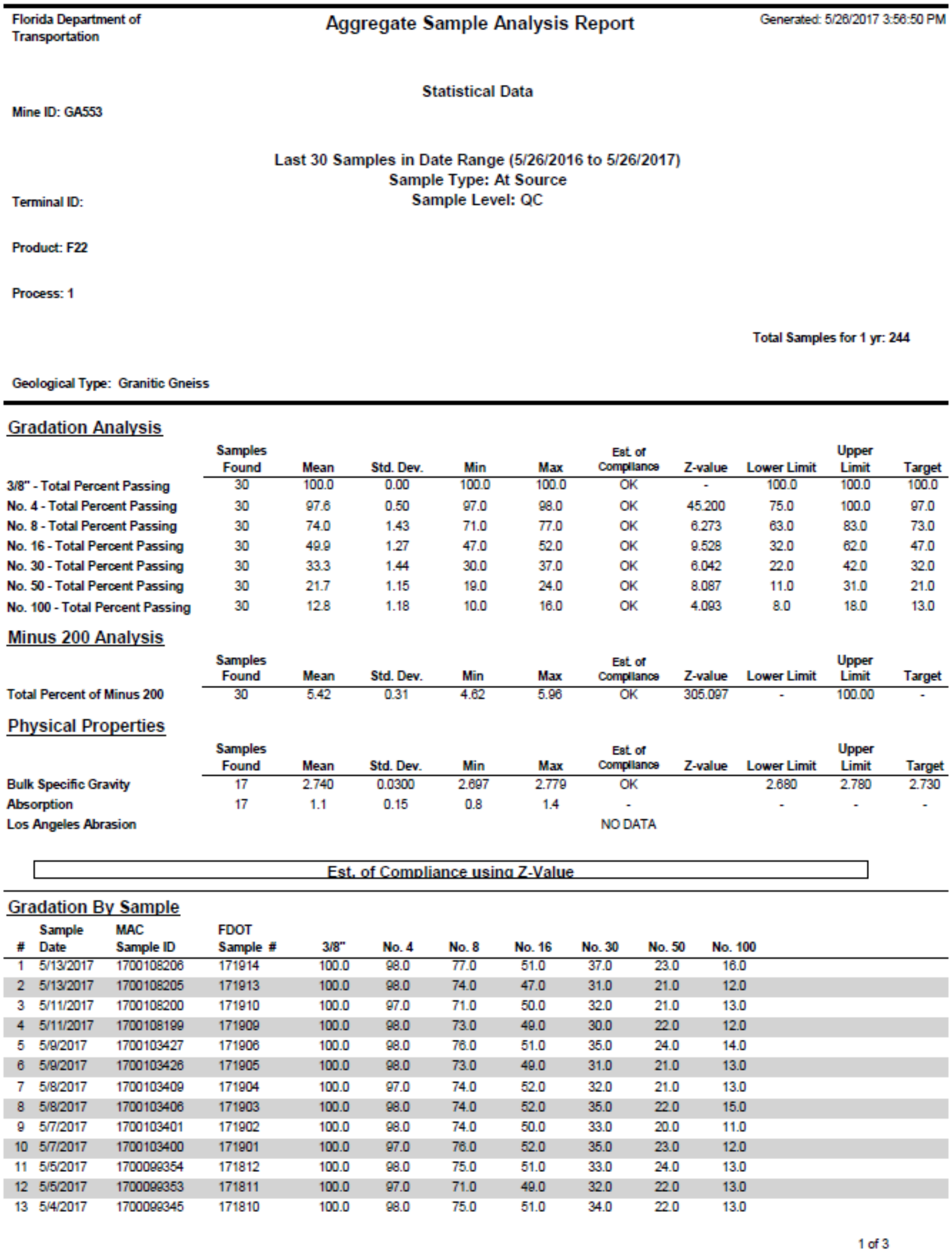
* Data provided by FDOT on June 5, 2017.

Specific Gravity # 78 Stone Granite

Geographic District	Managing District	Mine	Terminal	Facility Type	Product	Process	Description	Depleted?	Gsb
DISTRICT 2	DISTRICT 2	GA553	TM561	Terminal	C47	1	S1A Stone		2.775
DISTRICT 2	DISTRICT 2	GA553	TM759	Terminal	C47	1	S1A Stone		2.775

* Data provided by FDOT on June 5, 2017.

A. Granite w-10 Screenings



Florida Department of
Transportation

Aggregate Sample Analysis Report

Generated: 5/26/2017 3:56:50 PM

Statistical Data

Mine ID: GA553

Last 30 Samples in Date Range (5/26/2016 to 5/26/2017)
Sample Type: At Source
Sample Level: QC

Terminal ID:

Product: F22

Process: 1

Total Samples for 1 yr: 244

Geological Type: Granitic Gneiss

Gradation Analysis

	Samples Found	Mean	Std. Dev.	Min	Max	Est. of Compliance	Z-value	Lower Limit	Upper Limit	Target
3/8" - Total Percent Passing	30	100.0	0.00	100.0	100.0	OK	-	100.0	100.0	100.0
No. 4 - Total Percent Passing	30	97.6	0.50	97.0	98.0	OK	45.200	75.0	100.0	97.0
No. 8 - Total Percent Passing	30	74.0	1.43	71.0	77.0	OK	6.273	63.0	83.0	73.0
No. 16 - Total Percent Passing	30	49.9	1.27	47.0	52.0	OK	9.528	32.0	62.0	47.0
No. 30 - Total Percent Passing	30	33.3	1.44	30.0	37.0	OK	6.042	22.0	42.0	32.0
No. 50 - Total Percent Passing	30	21.7	1.15	19.0	24.0	OK	8.087	11.0	31.0	21.0
No. 100 - Total Percent Passing	30	12.8	1.18	10.0	16.0	OK	4.093	8.0	18.0	13.0

Minus 200 Analysis

	Samples Found	Mean	Std. Dev.	Min	Max	Est. of Compliance	Z-value	Lower Limit	Upper Limit	Target
Total Percent of Minus 200	30	5.42	0.31	4.62	5.96	OK	305.097	-	100.00	-

Physical Properties

	Samples Found	Mean	Std. Dev.	Min	Max	Est. of Compliance	Z-value	Lower Limit	Upper Limit	Target
Bulk Specific Gravity	17	2.740	0.0300	2.697	2.779	OK		2.680	2.780	2.730
Absorption	17	1.1	0.15	0.8	1.4	-		-	-	-
Los Angeles Abrasion						NO DATA				

Est. of Compliance using Z-Value

Gradation By Sample

#	Sample Date	MAC Sample ID	FDOT Sample #	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100
1	5/13/2017	1700108206	171914	100.0	98.0	77.0	51.0	37.0	23.0	16.0
2	5/13/2017	1700108205	171913	100.0	98.0	74.0	47.0	31.0	21.0	12.0
3	5/11/2017	1700108200	171910	100.0	97.0	71.0	50.0	32.0	21.0	13.0
4	5/11/2017	1700108199	171909	100.0	98.0	73.0	49.0	30.0	22.0	12.0
5	5/9/2017	1700103427	171906	100.0	98.0	76.0	51.0	35.0	24.0	14.0
6	5/9/2017	1700103426	171905	100.0	98.0	73.0	49.0	31.0	21.0	13.0
7	5/8/2017	1700103409	171904	100.0	97.0	74.0	52.0	32.0	21.0	13.0
8	5/8/2017	1700103406	171903	100.0	98.0	74.0	52.0	35.0	22.0	15.0
9	5/7/2017	1700103401	171902	100.0	98.0	74.0	50.0	33.0	20.0	11.0
10	5/7/2017	1700103400	171901	100.0	97.0	76.0	52.0	35.0	23.0	12.0
11	5/5/2017	1700099354	171812	100.0	98.0	75.0	51.0	33.0	24.0	13.0
12	5/5/2017	1700099353	171811	100.0	97.0	71.0	49.0	32.0	22.0	13.0
13	5/4/2017	1700099345	171810	100.0	98.0	75.0	51.0	34.0	22.0	13.0

1 of 3

14	5/4/2017	1700099344	171809	100.0	98.0	73.0	49.0	34.0	23.0	12.0
15	5/3/2017	1700100320	171808	100.0	98.0	74.0	50.0	34.0	23.0	13.0
16	5/3/2017	1700099343	171807	100.0	97.0	74.0	51.0	32.0	21.0	11.0
17	5/3/2017	1700100318	171807	100.0	98.0	74.0	50.0	34.0	22.0	13.0
18	5/2/2017	1700099337	171806	100.0	98.0	74.0	50.0	33.0	20.0	12.0
19	5/2/2017	1700099335	171805	100.0	97.0	71.0	47.0	32.0	19.0	10.0
20	5/1/2017	1700099330	171804	100.0	98.0	75.0	50.0	34.0	21.0	14.0
21	5/1/2017	1700099329	171803	100.0	97.0	73.0	49.0	34.0	22.0	13.0
22	4/30/2017	1700099324	171802	100.0	97.0	73.0	50.0	34.0	21.0	13.0
23	4/30/2017	1700099322	171801	100.0	97.0	75.0	50.0	32.0	22.0	13.0
24	4/25/2017	1700095439	171706	100.0	97.0	75.0	51.0	34.0	22.0	13.0
25	4/25/2017	1700095438	171705	100.0	98.0	74.0	50.0	34.0	21.0	12.0
26	4/24/2017	1700095431	171704	100.0	98.0	75.0	49.0	34.0	21.0	13.0
27	4/24/2017	1700095430	171703	100.0	97.0	75.0	50.0	34.0	22.0	14.0
28	4/23/2017	1700093467	171702	100.0	98.0	75.0	50.0	34.0	23.0	14.0
29	4/23/2017	1700093466	171701	100.0	97.0	75.0	49.0	33.0	21.0	13.0
30	4/20/2017	1700091795	171607	100.0	98.0	73.0	48.0	33.0	21.0	12.0

Minus 200 by Sample

#	Sample Date	MAC Sample ID	FDOT Sample #	Result
1	5/13/2017	1700108206	171914	5.60
2	5/13/2017	1700108205	171913	5.82
3	5/11/2017	1700108200	171910	5.66
4	5/11/2017	1700108199	171909	5.52
5	5/9/2017	1700103427	171906	5.96
6	5/9/2017	1700103426	171905	5.42
7	5/8/2017	1700103409	171904	5.38
8	5/8/2017	1700103406	171903	5.71
9	5/7/2017	1700103401	171902	5.53
10	5/7/2017	1700103400	171901	5.67
11	5/5/2017	1700099354	171812	5.32
12	5/5/2017	1700099353	171811	4.91
13	5/4/2017	1700099345	171810	5.56
14	5/4/2017	1700099344	171809	4.86
15	5/3/2017	1700100320	171808	5.65
16	5/3/2017	1700099343	171807	5.15
17	5/3/2017	1700100318	171807	5.41
18	5/2/2017	1700099337	171806	5.27
19	5/2/2017	1700099335	171805	4.62
20	5/1/2017	1700099330	171804	5.54
21	5/1/2017	1700099329	171803	4.86
22	4/30/2017	1700099324	171802	5.32
23	4/30/2017	1700099322	171801	5.19
24	4/25/2017	1700095439	171706	5.26
25	4/25/2017	1700095438	171705	5.60
26	4/24/2017	1700095431	171704	5.54
27	4/24/2017	1700095430	171703	5.39
28	4/23/2017	1700093467	171702	5.74
29	4/23/2017	1700093466	171701	5.52
30	4/20/2017	1700091795	171607	5.66

Start Weight by Sample

#	Sample Date	MAC Sample ID	FDOT Sample #	Result
1	5/13/2017	1700108206	171914	492.5
2	5/13/2017	1700108205	171913	539.5
3	5/11/2017	1700108200	171910	506.8
4	5/11/2017	1700108199	171909	546.9
5	5/9/2017	1700103427	171906	498.5
6	5/9/2017	1700103426	171905	509.4
7	5/8/2017	1700103409	171904	514.9
8	5/8/2017	1700103406	171903	504.1
9	5/7/2017	1700103401	171902	495.2
10	5/7/2017	1700103400	171901	541.8
11	5/5/2017	1700099354	171812	541.7
12	5/5/2017	1700099353	171811	509.2
13	5/4/2017	1700099345	171810	489.4
14	5/4/2017	1700099344	171809	526.7
15	5/3/2017	1700100320	171808	513.7
16	5/3/2017	1700099343	171807	487.6
17	5/3/2017	1700100318	171807	484.7
18	5/2/2017	1700099337	171806	462.9
19	5/2/2017	1700099335	171805	508.4
20	5/1/2017	1700099330	171804	471.4
21	5/1/2017	1700099329	171803	504.6
22	4/30/2017	1700099324	171802	497.8
23	4/30/2017	1700099322	171801	539.4
24	4/25/2017	1700095439	171706	516.9
25	4/25/2017	1700095438	171705	485.7
26	4/24/2017	1700095431	171704	494.7
27	4/24/2017	1700095430	171703	515.9
28	4/23/2017	1700093467	171702	484.7
29	4/23/2017	1700093466	171701	509.4
30	4/20/2017	1700091795	171607	501.9

Los Angeles Abrasion by Sample

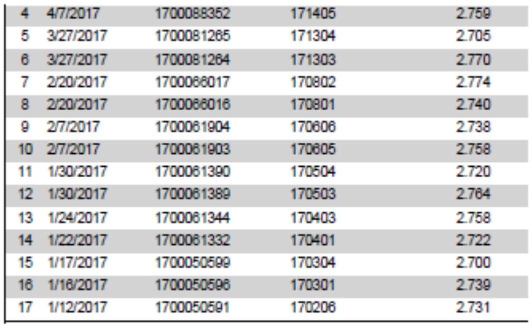
#	Sample Date	MAC Sample ID	FDOT Sample #	Result
1		NO DATA		

Bulk Specific Gravity by Sample

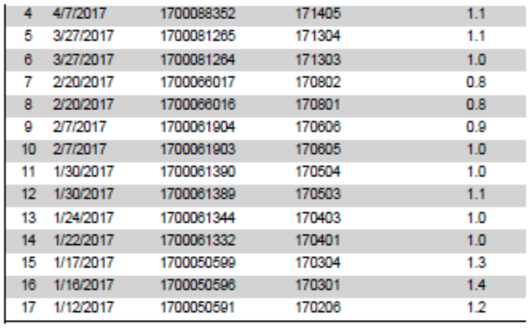
#	Sample Date	MAC Sample ID	FDOT Sample #	Result
1	4/10/2017	1700088373	171502	2.697
2	4/10/2017	1700088372	171501	2.779
3	4/7/2017	1700088353	171406	2.719

Absorption by Sample

#	Sample Date	MAC Sample ID	FDOT Sample #	Result
1	4/10/2017	1700088373	171502	1.1
2	4/10/2017	1700088372	171501	1.1
3	4/7/2017	1700088353	171406	1.0

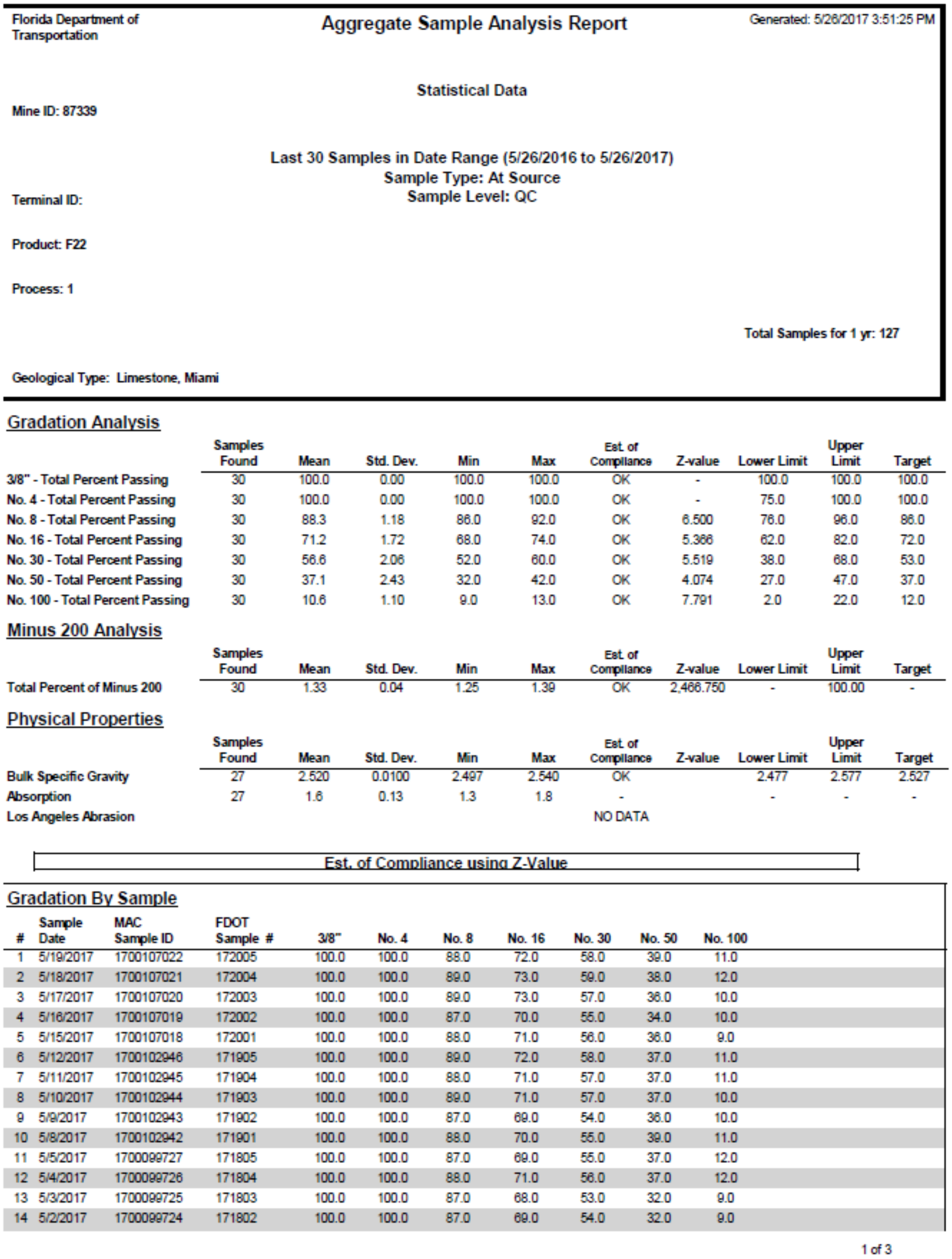


4	4/7/2017	1700088352	171405	2.759
5	3/27/2017	1700081265	171304	2.705
6	3/27/2017	1700081264	171303	2.770
7	2/20/2017	1700066017	170802	2.774
8	2/20/2017	1700066016	170801	2.740
9	2/7/2017	1700061904	170606	2.738
10	2/7/2017	1700061903	170605	2.758
11	1/30/2017	1700061390	170504	2.720
12	1/30/2017	1700061389	170503	2.764
13	1/24/2017	1700061344	170403	2.758
14	1/22/2017	1700061332	170401	2.722
15	1/17/2017	1700050599	170304	2.700
16	1/16/2017	1700050596	170301	2.739
17	1/12/2017	1700050591	170206	2.731



4	4/7/2017	1700088352	171405	1.1
5	3/27/2017	1700081265	171304	1.1
6	3/27/2017	1700081264	171303	1.0
7	2/20/2017	1700066017	170802	0.8
8	2/20/2017	1700066016	170801	0.8
9	2/7/2017	1700061904	170606	0.9
10	2/7/2017	1700061903	170605	1.0
11	1/30/2017	1700061390	170504	1.0
12	1/30/2017	1700061389	170503	1.1
13	1/24/2017	1700061344	170403	1.0
14	1/22/2017	1700061332	170401	1.0
15	1/17/2017	1700050599	170304	1.3
16	1/16/2017	1700050596	170301	1.4
17	1/12/2017	1700050591	170206	1.2

B. Limestone W-10 Screenings



Florida Department of Transportation

Aggregate Sample Analysis Report

Generated: 5/26/2017 3:51:25 PM

Statistical Data

Mine ID: 87339

Last 30 Samples in Date Range (5/26/2016 to 5/26/2017)
Sample Type: At Source
Sample Level: QC

Terminal ID:

Product: F22

Process: 1

Total Samples for 1 yr: 127

Geological Type: Limestone, Miami

Gradation Analysis

	Samples Found	Mean	Std. Dev.	Min	Max	Est. of Compliance	Z-value	Lower Limit	Upper Limit	Target
3/8" - Total Percent Passing	30	100.0	0.00	100.0	100.0	OK	-	100.0	100.0	100.0
No. 4 - Total Percent Passing	30	100.0	0.00	100.0	100.0	OK	-	75.0	100.0	100.0
No. 8 - Total Percent Passing	30	88.3	1.18	86.0	92.0	OK	6.500	76.0	96.0	86.0
No. 16 - Total Percent Passing	30	71.2	1.72	68.0	74.0	OK	5.366	62.0	82.0	72.0
No. 30 - Total Percent Passing	30	56.6	2.06	52.0	60.0	OK	5.519	38.0	68.0	53.0
No. 50 - Total Percent Passing	30	37.1	2.43	32.0	42.0	OK	4.074	27.0	47.0	37.0
No. 100 - Total Percent Passing	30	10.6	1.10	9.0	13.0	OK	7.791	2.0	22.0	12.0

Minus 200 Analysis

	Samples Found	Mean	Std. Dev.	Min	Max	Est. of Compliance	Z-value	Lower Limit	Upper Limit	Target
Total Percent of Minus 200	30	1.33	0.04	1.25	1.39	OK	2,466.750	-	100.00	-

Physical Properties

	Samples Found	Mean	Std. Dev.	Min	Max	Est. of Compliance	Z-value	Lower Limit	Upper Limit	Target
Bulk Specific Gravity	27	2.520	0.0100	2.497	2.540	OK		2.477	2.577	2.527
Absorption	27	1.6	0.13	1.3	1.8	-		-	-	-
Los Angeles Abrasion						NO DATA				

Est. of Compliance using Z-Value

Gradation By Sample

#	Sample Date	MAC Sample ID	FDOT Sample #	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100
1	5/19/2017	1700107022	172005	100.0	100.0	88.0	72.0	58.0	39.0	11.0
2	5/18/2017	1700107021	172004	100.0	100.0	89.0	73.0	59.0	38.0	12.0
3	5/17/2017	1700107020	172003	100.0	100.0	89.0	73.0	57.0	36.0	10.0
4	5/16/2017	1700107019	172002	100.0	100.0	87.0	70.0	55.0	34.0	10.0
5	5/15/2017	1700107018	172001	100.0	100.0	88.0	71.0	56.0	36.0	9.0
6	5/12/2017	1700102946	171905	100.0	100.0	89.0	72.0	58.0	37.0	11.0
7	5/11/2017	1700102945	171904	100.0	100.0	88.0	71.0	57.0	37.0	11.0
8	5/10/2017	1700102944	171903	100.0	100.0	89.0	71.0	57.0	37.0	10.0
9	5/9/2017	1700102943	171902	100.0	100.0	87.0	69.0	54.0	36.0	10.0
10	5/8/2017	1700102942	171901	100.0	100.0	88.0	70.0	55.0	39.0	11.0
11	5/5/2017	1700099727	171805	100.0	100.0	87.0	69.0	55.0	37.0	12.0
12	5/4/2017	1700099726	171804	100.0	100.0	88.0	71.0	56.0	37.0	12.0
13	5/3/2017	1700099725	171803	100.0	100.0	87.0	68.0	53.0	32.0	9.0
14	5/2/2017	1700099724	171802	100.0	100.0	87.0	69.0	54.0	32.0	9.0

1 of 3

15	5/1/2017	1700099723	171801	100.0	100.0	89.0	72.0	58.0	36.0	11.0
16	4/28/2017	1700099066	171705	100.0	100.0	89.0	72.0	56.0	34.0	9.0
17	4/27/2017	1700099065	171704	100.0	100.0	86.0	68.0	52.0	36.0	10.0
18	4/26/2017	1700099064	171703	100.0	100.0	87.0	69.0	56.0	39.0	12.0
19	4/25/2017	1700099063	171702	100.0	100.0	92.0	74.0	60.0	42.0	13.0
20	4/24/2017	1700099062	171701	100.0	100.0	88.0	70.0	55.0	35.0	11.0
21	4/21/2017	1700092486	171605	100.0	100.0	89.0	73.0	59.0	40.0	11.0
22	4/20/2017	1700092485	171604	100.0	100.0	90.0	73.0	56.0	35.0	9.0
23	4/19/2017	1700092484	171603	100.0	100.0	88.0	72.0	58.0	39.0	11.0
24	4/18/2017	1700092483	171602	100.0	100.0	89.0	72.0	59.0	39.0	11.0
25	4/17/2017	1700092482	171601	100.0	100.0	89.0	74.0	60.0	40.0	10.0
26	4/13/2017	1700091730	171504	100.0	100.0	89.0	73.0	59.0	41.0	12.0
27	4/12/2017	1700091729	171503	100.0	100.0	89.0	72.0	57.0	37.0	10.0
28	4/11/2017	1700091728	171502	100.0	100.0	89.0	73.0	59.0	40.0	10.0
29	4/10/2017	1700091727	171501	100.0	100.0	87.0	70.0	55.0	36.0	11.0
30	4/6/2017	1700084739	171404	100.0	100.0	89.0	71.0	56.0	37.0	9.0

Minus 200 by Sample

#	Sample Date	MAC Sample ID	FDOT Sample #	Result
1	5/19/2017	1700107022	172005	1.33
2	5/18/2017	1700107021	172004	1.35
3	5/17/2017	1700107020	172003	1.29
4	5/16/2017	1700107019	172002	1.38
5	5/15/2017	1700107018	172001	1.37
6	5/12/2017	1700102946	171905	1.34
7	5/11/2017	1700102945	171904	1.31
8	5/10/2017	1700102944	171903	1.28
9	5/9/2017	1700102943	171902	1.32
10	5/8/2017	1700102942	171901	1.39
11	5/5/2017	1700099727	171805	1.33
12	5/4/2017	1700099726	171804	1.36
13	5/3/2017	1700099725	171803	1.31
14	5/2/2017	1700099724	171802	1.30
15	5/1/2017	1700099723	171801	1.35
16	4/28/2017	1700099066	171705	1.25
17	4/27/2017	1700099065	171704	1.26
18	4/26/2017	1700099064	171703	1.35
19	4/25/2017	1700099063	171702	1.31
20	4/24/2017	1700099062	171701	1.29
21	4/21/2017	1700092486	171605	1.37
22	4/20/2017	1700092485	171604	1.34
23	4/19/2017	1700092484	171603	1.28
24	4/18/2017	1700092483	171602	1.31
25	4/17/2017	1700092482	171601	1.29
26	4/13/2017	1700091730	171504	1.35
27	4/12/2017	1700091729	171503	1.31
28	4/11/2017	1700091728	171502	1.38
29	4/10/2017	1700091727	171501	1.32
30	4/6/2017	1700084739	171404	1.35

Start Weight by Sample

#	Sample Date	MAC Sample ID	FDOT Sample #	Result
1	5/19/2017	1700107022	172005	489.1
2	5/18/2017	1700107021	172004	398.7
3	5/17/2017	1700107020	172003	417.8
4	5/16/2017	1700107019	172002	449.3
5	5/15/2017	1700107018	172001	409.9
6	5/12/2017	1700102946	171905	469.6
7	5/11/2017	1700102945	171904	451.0
8	5/10/2017	1700102944	171903	540.7
9	5/9/2017	1700102943	171902	400.3
10	5/8/2017	1700102942	171901	473.4
11	5/5/2017	1700099727	171805	480.5
12	5/4/2017	1700099726	171804	404.6
13	5/3/2017	1700099725	171803	465.2
14	5/2/2017	1700099724	171802	415.1
15	5/1/2017	1700099723	171801	488.3
16	4/28/2017	1700099066	171705	440.1
17	4/27/2017	1700099065	171704	397.1
18	4/26/2017	1700099064	171703	408.0
19	4/25/2017	1700099063	171702	436.5
20	4/24/2017	1700099062	171701	449.8
21	4/21/2017	1700092486	171605	481.4
22	4/20/2017	1700092485	171604	439.6
23	4/19/2017	1700092484	171603	413.6
24	4/18/2017	1700092483	171602	403.3
25	4/17/2017	1700092482	171601	502.7
26	4/13/2017	1700091730	171504	472.4
27	4/12/2017	1700091729	171503	532.9
28	4/11/2017	1700091728	171502	484.5
29	4/10/2017	1700091727	171501	400.6
30	4/6/2017	1700084739	171404	438.1

Los Angeles Abrasion by Sample

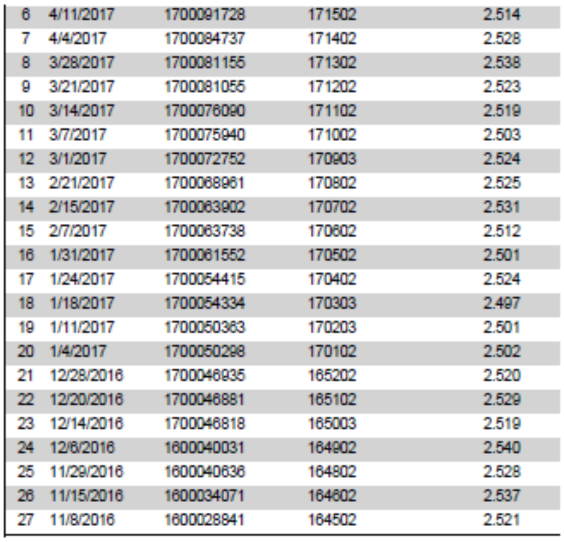
#	Sample Date	MAC Sample ID	FDOT Sample #	Result
1		NO DATA		

Bulk Specific Gravity by Sample

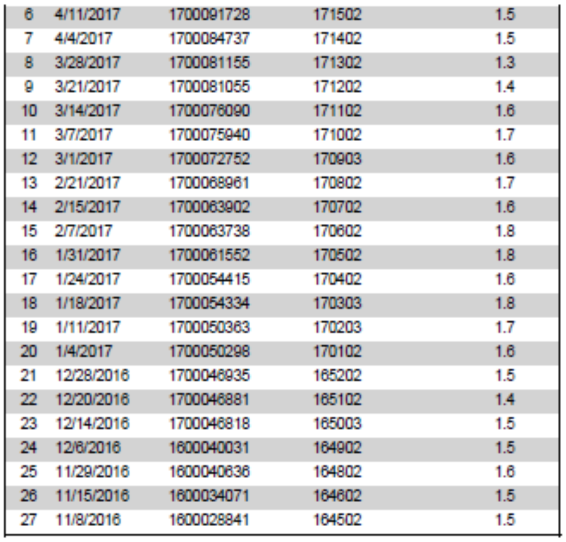
#	Sample Date	MAC Sample ID	FDOT Sample #	Result
1	5/16/2017	1700107019	172002	2.509
2	5/9/2017	1700102943	171902	2.527
3	5/3/2017	1700099725	171803	2.509
4	4/26/2017	1700099064	171703	2.508
5	4/18/2017	1700092483	171602	2.500

Absorption by Sample

#	Sample Date	MAC Sample ID	FDOT Sample #	Result
1	5/16/2017	1700107019	172002	1.6
2	5/9/2017	1700102943	171902	1.4
3	5/3/2017	1700099725	171803	1.6
4	4/26/2017	1700099064	171703	1.5
5	4/18/2017	1700092483	171602	1.7



6	4/11/2017	1700091728	171502	2.514
7	4/4/2017	1700084737	171402	2.528
8	3/28/2017	1700081155	171302	2.538
9	3/21/2017	1700081055	171202	2.523
10	3/14/2017	1700076090	171102	2.519
11	3/7/2017	1700075940	171002	2.503
12	3/1/2017	1700072752	170903	2.524
13	2/21/2017	1700068961	170802	2.525
14	2/15/2017	1700063902	170702	2.531
15	2/7/2017	1700063738	170602	2.512
16	1/31/2017	1700061552	170502	2.501
17	1/24/2017	1700054415	170402	2.524
18	1/18/2017	1700054334	170303	2.497
19	1/11/2017	1700050363	170203	2.501
20	1/4/2017	1700050298	170102	2.502
21	12/28/2016	1700046935	165202	2.520
22	12/20/2016	1700046881	165102	2.529
23	12/14/2016	1700046818	165003	2.519
24	12/6/2016	1600040031	164902	2.540
25	11/29/2016	1600040636	164802	2.528
26	11/15/2016	1600034071	164602	2.537
27	11/8/2016	1600028841	164502	2.521



6	4/11/2017	1700091728	171502	1.5
7	4/4/2017	1700084737	171402	1.5
8	3/28/2017	1700081155	171302	1.3
9	3/21/2017	1700081055	171202	1.4
10	3/14/2017	1700076090	171102	1.6
11	3/7/2017	1700075940	171002	1.7
12	3/1/2017	1700072752	170903	1.6
13	2/21/2017	1700068961	170802	1.7
14	2/15/2017	1700063902	170702	1.6
15	2/7/2017	1700063738	170602	1.8
16	1/31/2017	1700061552	170502	1.8
17	1/24/2017	1700054415	170402	1.6
18	1/18/2017	1700054334	170303	1.8
19	1/11/2017	1700050363	170203	1.7
20	1/4/2017	1700050298	170102	1.6
21	12/28/2016	1700046935	165202	1.5
22	12/20/2016	1700046881	165102	1.4
23	12/14/2016	1700046818	165003	1.5
24	12/6/2016	1600040031	164902	1.5
25	11/29/2016	1600040636	164802	1.6
26	11/15/2016	1600034071	164602	1.5
27	11/8/2016	1600028841	164502	1.5

APPENDIX C

PROPORTIONING OF AGGREGATE'S BLENDS

A. Hot Recycling

Limestone Virgin Aggregate Blend

Sieve Size			#78 Stone		W-10 Screenings		A + B	Limestone Blend	
			A (%)	50	B (%)	50	100	SP-19.0	
(in)	(mm)	^0.45	% Passing	A(%) * %Pass	% Passing	B(%) * %Pass	% Passing	Lower	Upper
1.5"	37.5	5.1	100.0	50.0	100.0	50.0	100.0		100
3/4"	19	3.8	100.0	50.0	100.0	50.0	100.0	90	100
1/2"	12.5	3.1	67.5	33.8	100.0	50.0	83.8		
3/8"	9.5	2.8	19.5	9.8	100.0	50.0	59.8		
#4	4.76	2	4.9	2.4	100.0	50.0	52.4		
#8	2.36	1.5	3.7	1.8	86.7	43.4	45.2	23	49
#16	1.19	1.1	3.2	1.6	64.4	32.2	33.8		
#30	0.595	0.8	3.0	1.5	45.5	22.8	24.3		
#50	0.297	0.6	2.7	1.3	19.2	9.6	10.9		
#100	0.149	0.4	2.1	1.0	4.9	2.5	3.5		
#200	0.074	0.3	1.4	0.7	0.7	0.4	1.1	2	8

Granite Virgin Aggregate Blend

Sieve Size			#78 Stone		W-10 Screenings		A + B	Granite Blend	
			A (%)	40	B (%)	60	100	SP-12.5	
(in)	(mm)	^0.45	% Passing	A(%) * %Pass	% Passing	B(%) * %Pass	% Passing	Lower	Upper
3/4"	19	3.8	100.0	40.0	100.0	60.0	100.0		100
1/2"	12.5	3.1	93.3	37.3	100.0	60.0	97.3	90	100
3/8"	9.5	2.8	49.4	19.8	100.0	60.0	79.8		
#4	4.76	2	11.3	4.5	97.2	58.3	62.8		
#8	2.36	1.5	3.7	1.5	72.0	43.2	44.7	28	58
#16	1.19	1.1	2.0	0.8	47.3	28.4	29.2		
#30	0.595	0.8	1.5	0.6	31.2	18.7	19.3		
#50	0.297	0.6	1.3	0.5	19.7	11.8	12.3		
#100	0.149	0.4	1.1	0.4	9.9	5.9	6.4		
#200	0.074	0.3	0.7	0.3	4.1	2.4	2.7	2	10

Aggregate Blend ABH-60L-L

Sieve Size			Limestone #78 Stone (C-41)		Limestone W-10 Screenings (F22)		Limestone RAP		A+B+C	SP-19	
			A (%)	**35**	**B (%)**	**5**	**C (%)**	**60**	**100**		
In	mm	^0.45	% Passing	A (%) %Pass	% Passing	B (%) %Pass	% Passing	C (%) %Pass	% Passing	Lower	Upper
1.5"	37.5	5.1	100.0	35.0	100.0	5.0	100.0	60.0	100.0		100.0
3/4"	19	3.8	100.0	35.0	100.0	5.0	100.0	60.0	100.0	90.0	100.0
1/2"	12.5	3.1	70.0	24.5	100.0	5.0	100.0	60.0	88.6		
3/8"	9.5	2.8	32.0	11.2	100.0	5.0	96.0	57.6	69.4		
#4	4.76	2	7.0	2.5	100.0	5.0	77.0	46.2	52.9		
#8	2.36	1.5	5.0	1.8	86.7	4.3	62.0	37.2	42.8	23.0	49.0
#16	1.19	1.1	5.0	1.8	64.4	3.2	53.0	31.8	36.2		
#30	0.595	0.8	4.0	1.4	45.5	2.3	46.0	27.6	30.9		
#50	0.297	0.6	4.0	1.4	19.2	1.0	33.0	19.8	21.7		
#100	0.149	0.4	3.0	1.1	4.9	0.2	18.0	10.8	11.8		
#200	0.074	0.3	3.0	1.1	0.7	0.0	8.5	5.1	5.6	2.0	8.0

Aggregate Blend ABH-60G-G

Sieve Size			Granite #78 Stone (C-47)		Granite W-10 Screenings (F22)		Granite/Limestone RAP		A+B+C	SP-12.5	
			A (%)	**20**	**B (%)**	**20**	**C (%)**	**60**	**100**		
In	mm	^0.45	% Passing	A (%) %Pass	% Passing	B (%) %Pass	% Passing	C (%) %Pass	% Passing	Lower	Upper
1.5"	37.5	5.1	100.0	20.0	100.0	20.0	100.0	60.0	100.0		100.0
3/4"	19	3.8	100.0	20.0	100.0	20.0	100.0	60.0	100.0	90.0	100.0
1/2"	12.5	3.1	93.3	18.7	100.0	20.0	97.0	58.2	88.6		
3/8"	9.5	2.8	49.4	9.9	100.0	20.0	93.0	55.8	69.4		
#4	4.76	2	11.3	2.3	97.2	19.4	71.0	42.6	52.9		
#8	2.36	1.5	3.7	0.7	72.0	14.4	49.0	29.4	42.8	23.0	49.0
#16	1.19	1.1	2.0	0.4	47.3	9.5	39.0	23.4	36.2		
#30	0.595	0.8	1.5	0.3	31.2	6.2	32.0	19.2	30.9		
#50	0.297	0.6	1.3	0.3	19.7	3.9	23.0	13.8	21.7		
#100	0.149	0.4	1.1	0.2	9.9	2.0	13.0	7.8	11.8		
#200	0.074	0.3	0.7	0.1	4.1	0.8	7.4	4.4	5.6	2.0	8.0

Aggregate Blend ABH-60L-G

Sieve Size			Granite #78 Stone (C-47)		Granite W-10 Screenings (F22)		Limestone RAP		A+B+C	SP-12.5	
			A (%)	**35**	**B (%)**	**5**	**C (%)**	**60**	**100%**		
In	mm	^0.45	% Passing	A (%) %Pass	% Passing	B (%) %Pass	% Passing	C (%) %Pass	% Passing	Lower	Upper
1.5"	37.5	5.1	100.0	35.0	100.0	5.0	100.0	60.0	100.0		
3/4"	19	3.8	100.0	35.0	100.0	5.0	100.0	60.0	100.0		100.0
1/2"	12.5	3.1	93.3	32.6	100.0	5.0	100.0	60.0	97.6	90.0	100.0
3/8"	9.5	2.8	49.4	17.3	100.0	5.0	96.0	57.6	79.9		
#4	4.76	2	11.3	4.0	97.2	4.9	77.0	46.2	55.0		
#8	2.36	1.5	3.7	1.3	72.0	3.6	62.0	37.2	42.1	28.0	58.0
#16	1.19	1.1	2.0	0.7	47.3	2.4	53.0	31.8	34.9		
#30	0.595	0.8	1.5	0.5	31.2	1.6	46.0	27.6	29.7		
#50	0.297	0.6	1.3	0.4	19.7	1.0	33.0	19.8	21.2		
#100	0.149	0.4	1.1	0.4	9.9	0.5	18.0	10.8	11.7		
#200	0.074	0.3	0.7	0.3	4.1	0.2	8.5	5.1	5.6	2.0	8.0

B. Cold Recycling

Aggregate Blend ABC-60L-LE or ABC-60L-LF

Sieve Size			Limestone #78 Stone (C-41)		Limestone W-10 Screenings (F22)		Limestone RAP		A+B+C	SP-19	
			A (%)	**25**	**B (%)**	**15**	**C (%)**	**60**	**100%**		
In	mm	^0.45	% Passing	A (%) %Pass	% Passing	B (%) %Pass	% Passing	C (%) %Pass	% Passing	Lower	Upper
1.5"	37.5	5.1	100.0	25.0	100.0	15.0	100.0	60.0	100.0		100.0
3/4"	19	3.8	100.0	25.0	100.0	15.0	100.0	60.0	100.0	90.0	100.0
1/2"	12.5	3.1	67.5	16.9	100.0	15.0	95.3	57.2	89.0		
3/8"	9.5	2.8	19.5	4.9	100.0	15.0	88.3	53.0	72.9		
#4	4.76	2	4.9	1.2	100.0	15.0	61.9	37.2	53.4		
#8	2.36	1.5	3.7	0.9	86.7	13.0	42.6	25.5	39.5	23.0	49.0
#16	1.19	1.1	3.2	0.8	64.4	9.7	30.1	18.1	28.6		
#30	0.595	0.8	3.0	0.7	45.5	6.8	20.1	12.0	19.6		
#50	0.297	0.6	2.7	0.7	19.2	2.9	8.6	5.2	8.7		
#100	0.149	0.4	2.1	0.5	4.9	0.7	1.6	0.9	2.2		
#200	0.074	0.3	1.4	0.3	0.7	0.1	0.2	0.1	0.6	2.0	8.0

Aggregate Blend ABC-60G-GE

Sieve Size			**Granite #78 Stone (C-47)**		**Granite W-10 Screenings (F22)**		**Limestone/ Granite RAP**		**A+B+C**	**SP-12.5**	
			A (%)	**5**	**B (%)**	**35**	**C (%)**	**60**	**100%**		
In	mm	^0.45	% Passing	A (%) %Pass	% Passing	B (%) %Pass	% Passing	C (%) %Pass	% Passing	Lower	Upper
1.5"	37.5	5.1	100.0	5.0	100.0	35.0	100.0	60.0	100.0		
3/4"	19	3.8	100.0	5.0	100.0	35.0	95.4	57.2	97.2		100.0
1/2"	12.5	3.1	93.3	4.7	100.0	35.0	84.3	50.6	90.2	90.0	100.0
3/8"	9.5	2.8	49.4	2.5	100.0	35.0	72.5	43.5	81.0		
#4	4.76	2	11.3	0.6	97.2	34.0	43.3	26.0	60.5		
#8	2.36	1.5	3.7	0.2	72.0	25.2	23.7	14.2	39.6	28.0	58.0
#16	1.19	1.1	2.0	0.1	47.3	16.6	12.2	7.3	24.0		
#30	0.595	0.8	1.5	0.1	31.2	10.9	2.7	1.6	12.6		
#50	0.297	0.6	1.3	0.1	19.7	6.9	0.3	0.2	7.1		
#100	0.149	0.4	1.1	0.1	9.9	3.5	0.0	0.0	3.5		
#200	0.074	0.3	0.7	0.0	4.1	1.4	0.0	0.0	1.5	2.0	8.0

Aggregate Blend ABC-60L-GF

Sieve Size			**Granite #78 Stone (C-47)**		**Granite W-10 Screenings (F22)**		**Limestone RAP**		**A+B+C**	**SP-12.5**	
			A (%)	**20**	**B (%)**	**20**	**C (%)**	**60**	**100%**		
In	mm	^0.45	% Passing	A (%) %Pass	% Passing	B (%) %Pass	% Passing	C (%) %Pass	% Passing	Lower	Upper
1.5"	37.5	5.1	100.0	20.0	100.0	20.0	100.0	60.0	100.0		
3/4"	19	3.8	100.0	20.0	100.0	20.0	100.0	60.0	100.0		100.0
1/2"	12.5	3.1	93.3	18.7	100.0	20.0	95.3	57.2	95.8	90.0	100.0
3/8"	9.5	2.8	49.4	9.9	100.0	20.0	88.3	53.0	82.9		
#4	4.76	2	11.3	2.3	97.2	19.4	61.9	37.2	58.9		
#8	2.36	1.5	3.7	0.7	72.0	14.4	42.6	25.5	40.7	28.0	58.0
#16	1.19	1.1	2.0	0.4	47.3	9.5	30.1	18.1	27.9		
#30	0.595	0.8	1.5	0.3	31.2	6.2	20.1	12.0	18.6		
#50	0.297	0.6	1.3	0.3	19.7	3.9	8.6	5.2	9.4		
#100	0.149	0.4	1.1	0.2	9.9	2.0	1.6	0.9	3.1		
#200	0.074	0.3	0.7	0.1	4.1	0.8	0.2	0.1	1.1	2.0	8.0

APPENDIX D

BINDER PG 52-28 PG GRADE TEST RESULTS

A. Replicate 1

Property	**PG 52-28**		
Original Properties			
Dynamic Shear			
Min. 1.0 kPa	G*/sinδ at 52ºC	1.98	kPa
	G*/ sinδ at 58ºC	0.86	kPa
Rolling Thin Film Oven (RTFO) Aged Binder			
Dynamic Shear			
Min. 2.2 kPa	G*/ sinδ at 52ºC	4.84	kPa
	G*/ sinδ at 58ºC	1.98	kPa
Rolling Thin film Oven (RTFO) and PAV Aged Binder			
Dynamic Shear			
Max. 5000 kPa	G* sinδ at 13ºC	7345	kPa
	G* sinδ at 16ºC	4779	kPa
Creep Stifness			
S. Max. 300 MPa	Temperature	S (MPa)	m-Value (-)
m-Value Min. 0.3	-18ºC	211	0.345
	-24ºC	400	0.268

B. Replicate 2

Property	**PG 52-28**		
Original Properties			
Dynamic Shear			
Min. 1.0 kPa	G*/ sinδ at 52ºC	1.98	kPa
	G*/ sinδ at 58ºC	0.85	kPa
Rolling Thin Film Oven (RTFO) Aged Binder			
Dynamic Shear			
Min. 2.2 kPa	G*/ sinδ at 52ºC	4.84	kPa
	G*/ sinδ at 58ºC		kPa
Rolling Thin film Oven (RTFO) and PAV Aged Binder			
Dynamic Shear			
Max. 5000 kPa	G* sinδ at 13ºC	7234	kPa
	G* sinδ at 16ºC	4701	kPa
Creep Stiffness			
S. Max. 300 MPa	Temperature	S (MPa)	m-Value (-)
m-Value Min. 0.3	-18ºC	193	0.347
	-24ºC	435	0.264

APPENDIX E

BINDER CONTENT OF RAP SOURCES

A. Calibration Factors

	Limestone Mixture			**Granite Mixture**	
Sample	**1**	**2**	**3**	**1**	**2**
AC_{Actual} (%)	4.5%	4.5%	7.0%	4.5%	4.5%
Basket Mass (g)	3046.5	3050.7	3045.0	3043.7	3045.2
Basket + Sample Mass (g)	5348.7	5272.2	5032.5	5199.7	5372.6
Initial Sample Mass (g)	2302.2	2221.5	1987.5	2156.0	2327.4
Basket + Sample Mass (g) - After	5240.5	5166.4	4888.1	5096.7	5261.4
Final Sample Mass (g)	2194.0	2115.7	1843.1	2053.0	2216.2
Mass Loss (g)	108.2	105.8	144.4	103.0	111.2
$AC_{Measured}$ (%)	4.70%	4.76%	7.27%	4.78%	4.78%
W_L (%)	-0.20%	-0.26%	-0.27%	-0.28%	-0.28%
CF[AC]	-0.24%			-0.28%	

B. Binder Contents

	Stockpile 1-09: Limestone Aggregate		**Stockpile 1-16: Granite/Limestone Aggregate**	
Sample	**1**	**2**	**1**	**2**
Basket Mass (g)	3042.0	2850.4	2852.0	2850.5
Basket + Sample Mass (g)	5078.5	5999.9	5347.3	6060.1
Initial Sample Mass (g)	2036.5	3149.5	2495.3	3209.6
Basket + Sample Mass (g) - After	4963.5	5825.6	5222.2	5897.5
Final Sample Mass (g)	1921.5	2975.2	2370.2	3047.0
Mass Loss (g)	115.0	174.3	125.1	162.6
$AC_{Measured}$ (%)	5.65%	5.53%	5.01%	5.07%
CF[AC]	-0.24%		-0.28%	
$AC_{Calibrated}$ (%)	5.40%	5.29%	4.74%	4.79%
Average $AC_{Calibrated}$ (%)	5.35%		4.76%	

APPENDIX F

RBR ESTIMATION

A. Limestone + RAP Mixture

MIX

Virgin Aggregate	Limestone (C-41)

RAP	
RAP Source	STK 09 - Limestone RAP
RAP Content of The Mix (%)	60
Binder Content of RAP (%)	5.4

Virgin Binder	
Binder	PG 52-28
Optimum Binder Content, OBC (%)	6.8

$$RBR = \frac{60.0\% * 5.4\%}{6.8\%} = 0.48$$

B. Granite + RAP Mixture

MIX

Virgin Aggregate	Granite (C-47)

RAP	
RAP Source	STK 16 - Limestone/Granite RAP
RAP Content of The Mix (%)	60
Binder Content of RAP (%)	4.8

Virgin Binder	
Binder	PG 52-28
Optimum Binder Content, OBC (%)	6

$$RBR = \frac{60.0\% * 4.8\%}{6.0\%} = 0.48$$

APPENDIX G

RECYCLING AGENT SELECTION TEST RESULTS

A. Rheological Characterization

RAP	Recycling Agent	Recycling Agent Dosage (%)	High Temperature PG						PGH Change %
			Unaged			RTFO + PAV40			
			Rep. 1	Rep. 2	Average	Rep. 1	Rep. 2	Average	
Stockpile 1-09	B-2	5.1	65.4	64.5	64.95	83.6	83.9	83.75	28.95
	B-1		65.7	65.8	65.75	82.2	82.3	82.25	25.10
	P-2		70.5	70.3	70.40	85.8	85.8	85.80	21.88
	P-1		70	69.8	69.90	85.9	85.8	85.85	22.82

RAP	Recycling Agent	Recycling Agent Dosage (%)	High Temperature PG						PGH Change %
			Original			RTFO + PAV40			
			Rep. 1	Rep. 2	Average	Rep. 1	Rep. 2	Average	
Stockpile 1-16	B-2	5.9	67.6	67.7	67.65	83.6	83.4	83.50	23.43
	B-1		67.5	67.5	67.50	85.2	85.3	85.25	26.30
	P-2		72.3	72.1	72.20	89.3	89.1	89.20	23.55
	P-1		71.7	71.8	71.75	88.4	88.1	88.25	23.00

B. Chemical Characterization

Recycling Agent	RAP	Carbonyl Area (TAMU Method (Glover et al., 2007)) (-)											CA Change%
		Unaged					RTFO + PAV40						
		R. 1	R. 2	R. 3	R. 4	Average	R. 1	R. 2	R. 3	R. 4	R. 5	Average	
B-2	Stockpile 1-16	1.69	1.74	1.75	1.78	1.74	2.35	2.20	2.29			2.28	31.2
B-1		1.86	1.85	1.78		1.83	2.44	2.41	1.65	2.49	2.49	2.30	25.7
P-2		1.28	1.29	1.24		1.27	1.90	1.91	1.87			1.89	49.0
P-1		1.28	1.18	1.23		1.23	1.80	1.81	1.88			1.83	48.9

Recycling Agent	RAP	Carbonyl Area (TAMU Method (Glover et al., 2007)) (-)								CA Change%
		Unaged				RTFO + PAV40				
		Rep. 1	Rep. 2	Rep. 3	Average	Rep. 1	Rep. 2	Rep. 3	Average	
B-2	Stockpile 1-09	1.63	1.51	1.52	1.55	2.19	2.20	2.08	2.16	39.0
B-1		1.63	1.67	1.61	1.64	2.20	2.16	2.20	2.19	33.6
P-2		1.21	1.22	1.19	1.20	1.72	1.72	1.72	1.72	42.8
P-1		1.13	1.16	1.14	1.14	1.61	1.65	1.62	1.63	42.1

APPENDIX H

RECYCLING AGENT DOSAGE VERIFICATION RESULTS

A. RAP Binder Stockpile 1-09 Blends

Recycling Agent Dosage (%)	**High Temperature PG**							
	B-1				**P-1**			
	Unaged G*/sin(δ)		**RTFO G*/sin(δ)**		**Unaged G*/sin(δ)**		**RTFO G*/sin(δ)**	
0.0	74.8	74.4	76	76.3	74.8	74.5	75.4	75.6
2.0	72.6	72.2	73.6	73.8	72.3	72.1	72.9	73
8.0	62	62.1	63.2	63.9	66.9	66.8	67.8	67.6

Recycling Agent Dosage (%)	**High Temperature PG**			
	B-1		**Hydrolene**	
	Average Unaged G*/sin(δ)	**Average RTFO G*/sin(δ)**	**Average Unaged G*/sin(δ)**	**Average RTFO G*/sin(δ)**
	OB	**RTFO**	**OB**	**RTFO**
0.0	74.6	76.2	74.7	75.5
2.0	72.4	73.7	72.2	73.0
8.0	62.1	63.6	66.9	67.7

B. RAP Binder Stockpile 1-16 Blends

Recycling Agent Dosage (%)	**High Temperature PG Grade**							
	B-1				**P-1**			
	Unaged G*/sin(δ)		**RTFO G*/sin(δ)**		**Unaged G*/sin(δ)**		**RTFO G*/sin(δ)**	
0.0	77.5	77.7	78.5	78.6	77.5	77.9	78.8	78.9
2.0	73.4	73.4	74.9	74.9	74.9	74.8	76.3	76.1
8.0	65.2	65.4	65.2	64.9	70.1	69.9	71.2	71.4
14.0	-		-		65.1	64.7	66.3	66.4

Recycling Agent Dosage (%)	**High Temperature PG Grade**			
	B-1		**P-1**	
	Average Unaged G*/sin(δ)	**Average RTFO G*/sin(δ)**	**Average Unaged G*/sin(δ)**	**Average RTFO G*/sin(δ)**
	OB	**RTFO**	**OB**	**RTFO**
0.0	77.6	78.6	77.7	78.9
2.0	73.4	74.9	74.9	76.2
8.0	65.3	65.1	70.0	71.3
14.0	-	-	64.9	66.4

APPENDIX I

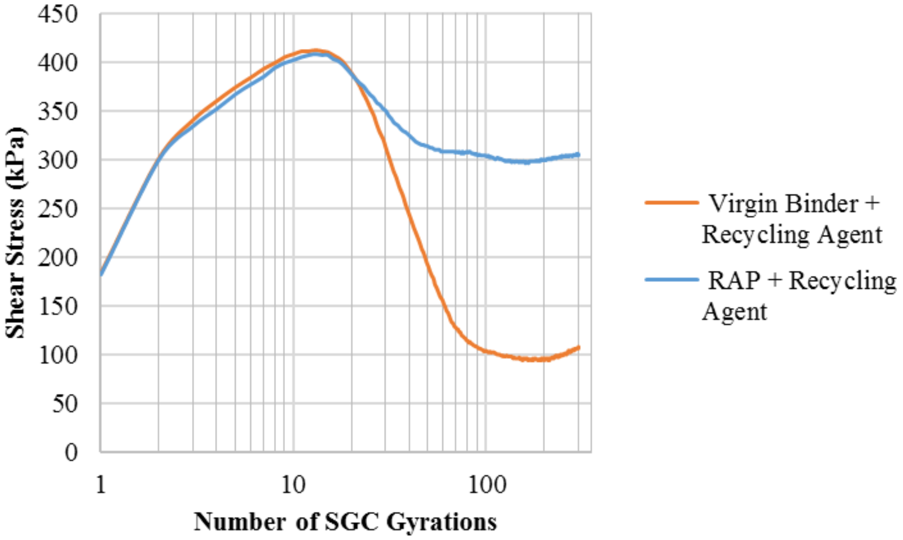
RECYCLING AGENT ADDITION METHOD TEST RESULTS

A. Workability

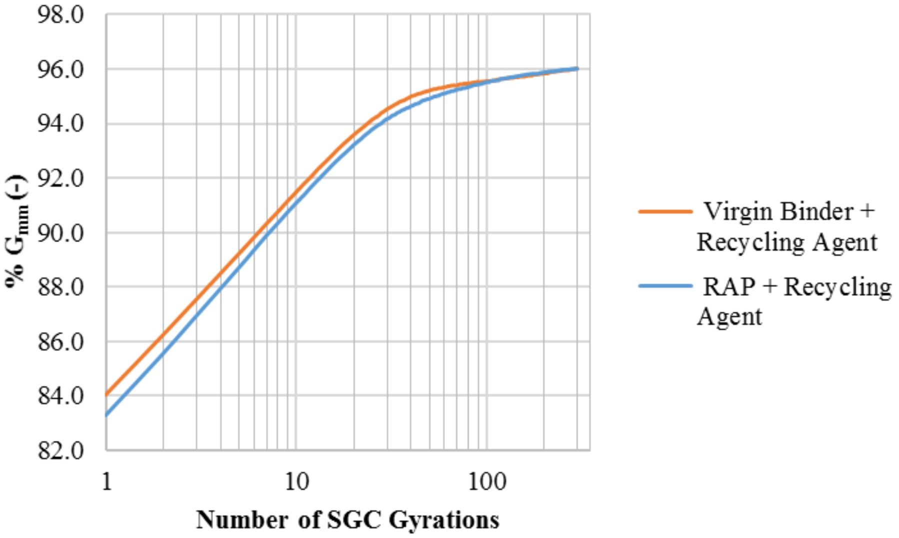
Organic Based Recycling Agent: B-1

- Determination of Maximum Specific Gravity (G_{mm}):

Aggregates	Sample	P_b (%)	$W_{mix\text{-}loose}$ (g)	$W_{pyc\ (soak)}$ (g)	$W_{spyc+mix\ (soak)}$ (g)	G_{mm} (-)
Granite	1	5.9	1848.3	1569.7	2676.8	2.494
	2	5.9	1844.8	1571.7	2669.3	2.469
					Average	2.481



B-1 Workability Test Results – Shear Stress Evolution

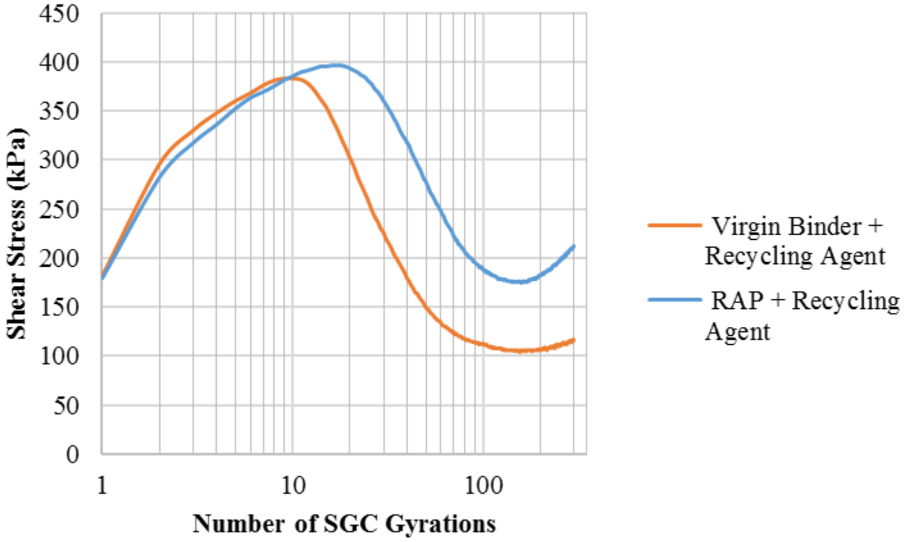


B-1 Workability Test Results – G_{mm} Evolution

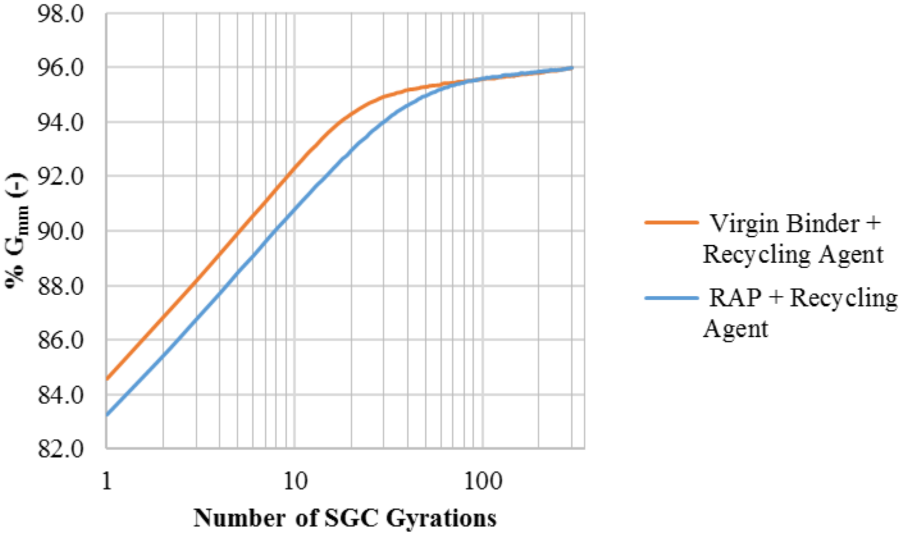
Petroleum Based Recycling Agent: P-1

- Determination of Maximum Specific Gravity (G_{mm}):

Aggregates	Sample	P_b (%)	$W_{mix\text{-}loose}$ (g)	$W_{pyc\ (soak)}$ (g)	$W_{spyc+mix\ (soak)}$ (g)	G_{mm} (-)
Granite	1	5.9	1862.8	1569.7	2673.1	2.453
	2	5.9	1829.1	1571.7	2660.5	2.471
					Average	2.462



P-1 Workability Test Results – Shear Stress Evolution



P-1 Workability Test Results – G_{mm} Evolution

B. Coatability

Organic Based Recycling Agent: B-1

- Virgin binder + Recycling Agent

W_{agg} OD-1 (g)	2000.0	W_{loose} OD-1 (g)	986.1	W_{loose} SSD-1 (g)	986.5
		W_{loose} OD-2(g)	985.3	W_{loose} SSD-2 (g)	985.4
W_{agg} OD-2 (g)	2001.6			W_{agg} SSD-2 (g)	2009.0

Absorption_{Agg} (%)	0.37
$\text{Absorption}_{Loose\text{-}1}$ (%)	0.04
$\text{Absorption}_{Loose\text{-}2}$ (%)	0.01
$\text{Absorption}_{Loose\text{-}average}$ (%)	0.03

Coatability Index, CI (%)	93.1

- RAP + Recycling Agent

W_{agg} OD-1 (g)	2000.0	W_{loose} OD-1 (g)	975.5	W_{loose} SSD-1 (g)	976.1
		W_{loose} OD-2(g)	973.7	W_{loose} SSD-2 (g)	974.4
W_{agg} OD-2 (g)	1997.0			W_{agg} SSD-2 (g)	2005.3

Absorption_{Agg} (%)	0.42
$\text{Absorption}_{Loose\text{-}1}$ (%)	0.06
$\text{Absorption}_{Loose\text{-}2}$ (%)	0.07
$\text{Absorption}_{Loose\text{-}average}$ (%)	0.07

Coatability Index, CI (%)	84.0

Petroleum Based Recycling Agent: P-1

- Virgin binder + Recycling Agent

W_{agg} OD-1 (g)	2000.0	W_{loose} OD-1 (g)	977.3	W_{loose} SSD-1 (g)	978.2
		W_{loose} OD-2(g)	974.2	W_{loose} SSD-2 (g)	974.2
W_{agg} OD-2 (g)	2002.0			W_{agg} SSD-2 (g)	2011.8

- Virgin binder + Recycling Agent (continued)

Absorption$_{\text{Agg}}$ (%)	0.5
Absorption$_{\text{Loose-1}}$ (%)	0.1
Absorption$_{\text{Loose-2}}$ (%)	0.0
Absorption$_{\text{Loose-average}}$ (%)	0.0

Coatability Index, CI (%)	90.6

- RAP + Recycling Agent

W$_{\text{agg}}$ OD-1 (g)	2000.0	W$_{\text{loose}}$ OD-1 (g)	974.6	W$_{\text{loose}}$ SSD-1 (g)	975.6
		W$_{\text{loose}}$ OD-2(g)	975.8	W$_{\text{loose}}$ SSD-2 (g)	976.4
W$_{\text{agg}}$ OD-2 (g)	1990.4			W$_{\text{agg}}$ SSD-2 (g)	1997.1

Absorption$_{\text{Agg}}$ (%)	0.3
Absorption$_{\text{Loose-1}}$ (%)	0.1
Absorption$_{\text{Loose-2}}$ (%)	0.1
Absorption$_{\text{Loose-average}}$ (%)	0.1

Coatability Index, CI (%)	75.6

APPENDIX J

MIX DESIGN VOLUMETRIC CALCULATIONS

A. Hot Recycling – Virgin Mixtures

Limestone Virgin Mixture Design

- Determination Bulk Specific Gravity (Gsb)

P_{b-Wagg} (%)	**6.9**
P_b (%)	**6.5**

Sample	**W_{Dry} (g)**	**W_{soak} (g)**	**W_{SSD} (g)**	**G_{mb} (-)**	**G_{mm} (-)**	**H @ N_{Ini} (mm)**	**H @ N_{des} (mm)**
1	4528.00	2533.30	4537.00	2.260	2.358	123.0	115.4
2	4507.30	2500.70	4522.90	2.229		123.8	116.0
			Average	2.245		123.4	115.7

P_{b-Wagg} (%)	**7.5**
P_b (%)	**7.0**

Sample	**W_{Dry} (g)**	**W_{soak} (g)**	**W_{SSD} (g)**	**G_{mb} (-)**	**G_{mm} (-)**	**H @ N_{Ini} (mm)**	**H @ N_{des} (mm)**
1	4504.80	2517.60	4508.80	2.262	2.341	122.5	114.6
2	4483.80	2513.30	4492.20	2.266		122.9	114.9
			Average	2.264		122.7	114.8

P_{b-Wagg} (%)	**8.1**
P_b (%)	**7.5**

Sample	**W_{Dry} (g)**	**W_{soak} (g)**	**W_{SSD} (g)**	**G_{mb} (-)**	**G_{mm} (-)**	**H @ N_{Ini} (mm)**	**H @ N_{des} (mm)**
1	4506.60	2517.30	4509.00	2.263	2.324	122.3	114.6
2	4506.80	2529.50	4508.90	2.277		122.4	114.4
			Average	2.270		122.3	114.5

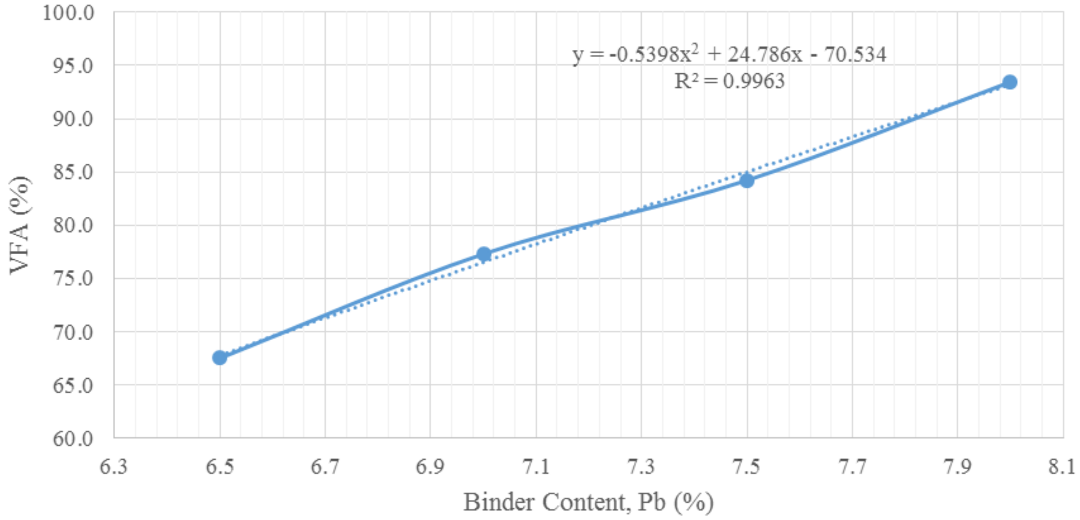
P_{b-Wagg} (%)	**8.7**
P_b (%)	**8.0**

Sample	**W_{Dry} (g)**	**W_{soak} (g)**	**W_{SSD} (g)**	**G_{mb} (-)**	**G_{mm} (-)**	**H @ N_{Ini} (mm)**	**H @ N_{des} (mm)**
1	4504.30	2528.10	4505.70	2.278	2.308	122.2	113.8
2	4512.60	2546.10	4513.90	2.293		121.5	113.2
			Average	2.286		121.9	113.5

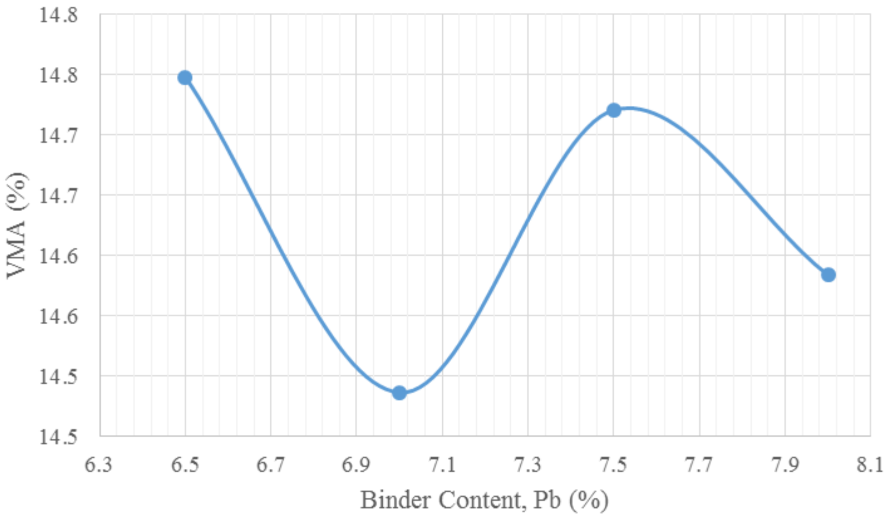
- Determination of Maximum Specific Gravity (Gmm) and Effective Specific Gravity (Gse)

Aggregates	Sample	P_b (%)	$W_{mix\text{-}loose}$ (g)	$W_{pyc\ (soak)}$ (g)	$W_{spyc+mix\ (soak)}$ (g)	G_{mm} (-)	G_{se} (-)
Limestone	1	7.5	1829.4	1571.9	2616.9	2.3322	2.615
	2	7.5	1831.1	1570.0	2610.4	2.3158	2.592

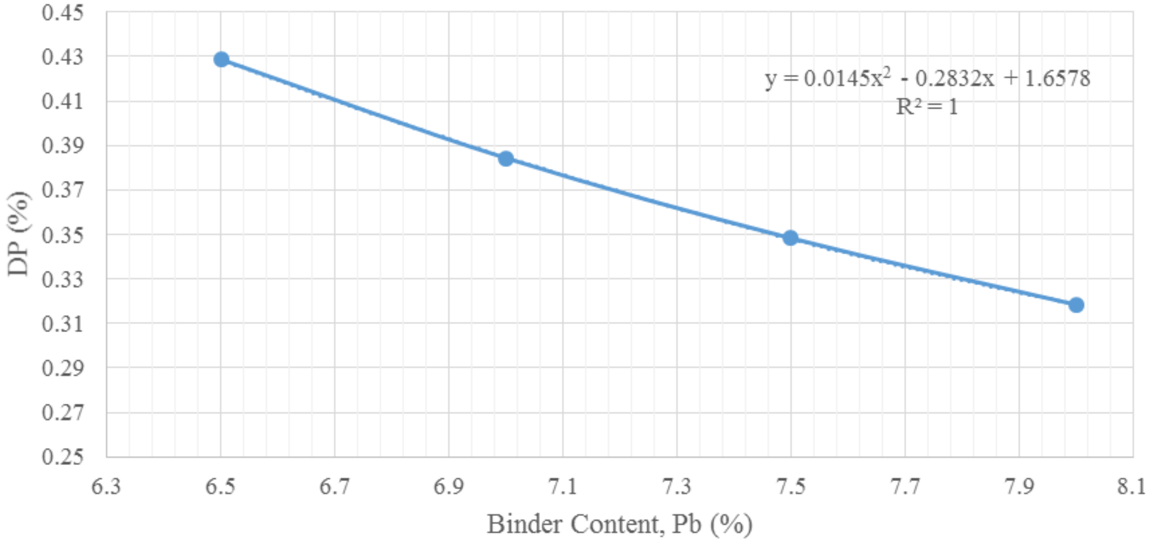
- Volumetric Properties



VFA Results Limestone Mixture



VMA Results Limestone Mixture



Dust Proportion (DP) Results Limestone Mixture

Granite Virgin Mixture Mixture

- Determination Bulk Specific Gravity (Gsb)

$P_{b\text{-}Wagg}$ (%)	**4.4**						
P_b (%)	**5.0**						
Sample	**W_{Dry} (g)**	**W_{soak} (g)**	**W_{SSD} (g)**	**G_{mb} (-)**	**G_{mm} (-)**	**H @ N_{Ini} (mm)**	**H @ N_{des} (mm)**
1	4518.40	2645.50	4541.70	2.383	2.583	118.0	108.9
2	4512.40	2653.90	4533.70	2.400		117.1	108.3
			Average	2.392		117.5	108.6

$P_{b\text{-}Wagg}$ (%)	**4.9**						
P_b (%)	**5.5**						
Sample	**W_{Dry} (g)**	**W_{soak} (g)**	**W_{SSD} (g)**	**G_{mb} (-)**	**G_{mm} (-)**	**H @ N_{Ini} (mm)**	**H @ N_{des} (mm)**
1	4521.40	2657.30	4531.20	2.413	2.562	117.6	108.3
2	4512.30	2660.90	4522.20	2.424		116.8	107.7
			Average	2.419		117.2	108.0

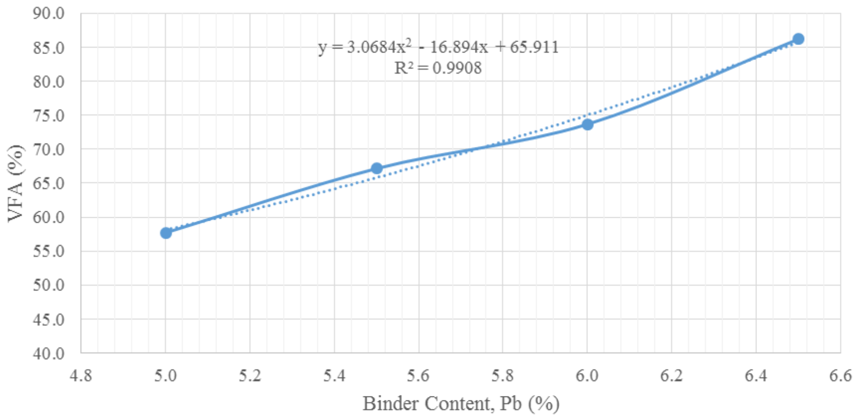
$P_{b\text{-}Wagg}$ (%)	**6.4**						
P_b (%)	**6.0**						
Sample	**W_{Dry} (g)**	**W_{soak} (g)**	**W_{SSD} (g)**	**G_{mb} (-)**	**G_{mm} (-)**	**H @ N_{Ini} (mm)**	**H @ N_{des} (mm)**
1	4519.10	2658.60	4525.30	2.421	2.541	117.4	107.9
2	4513.00	2662.60	4518.70	2.431		117.0	107.4
			Average	2.426		117.2	107.6

$P_{b\text{-}Wagg}$ (%)	**7.0**						
P_b (%)	**6.5**						
Sample	**W_{Dry} (g)**	**W_{soak} (g)**	**W_{SSD} (g)**	**G_{mb} (-)**	**G_{mm} (-)**	**H @ N_{Ini} (mm)**	**H @ N_{des} (mm)**
1	4512.20	2680.90	4514.10	2.461	2.520	115.1	105.8
2	4519.40	2686.60	4521.00	2.464		115.9	106.1
			Average	2.463		115.5	106.0

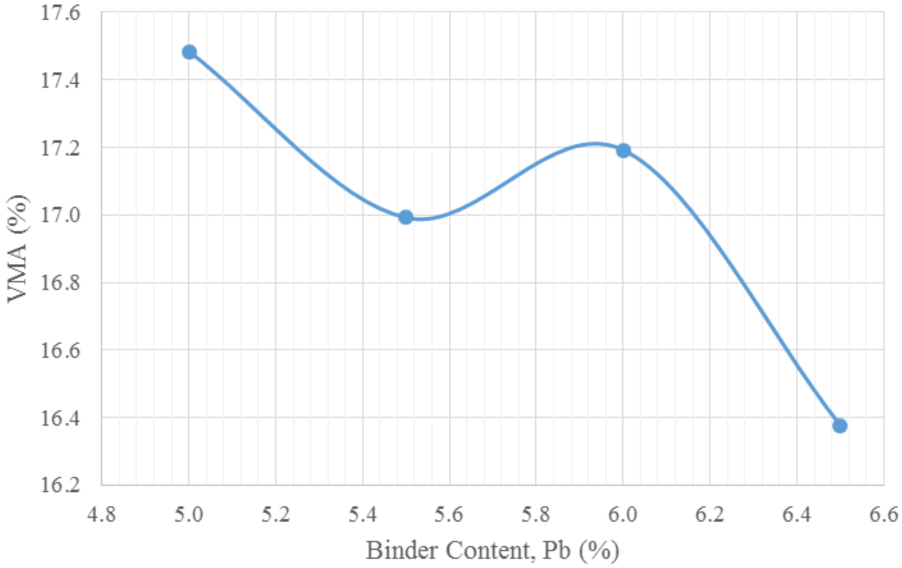
- Determination of Maximum Specific Gravity (Gmm) and Effective Specific Gravity (Gse)

Aggregates	**Sample**	**P_b (%)**	**$W_{mix\text{-}loose}$ (g)**	**$W_{pyc\ (soak)}$ (g)**	**$W_{spyc+mix\ (soak)}$ (g)**	**G_{mm} (-)**	**G_{se} (-)**
Granite	1	5.0	1831.4	1572	2694.7	2.5842	2.819
	2	5.0	1815.6	1569.8	2682.4	2.5826	2.817

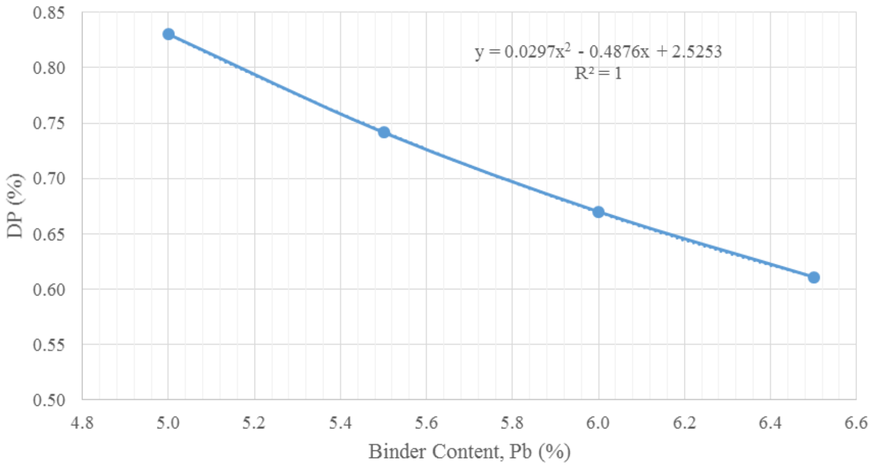
- Volumetric Properties



VFA Results Granite Mixture



VMA Results Granite Mixture



Dust Proportion (DP) Results Granite Mixture

B. Cold Recycling with Emulsified Asphalt

C-100L-E Mixture

Conditioning	Residual Binder Content (%)	Sample	Height (mm)					Peak Load (kN)	Tensile Strength, S_t (kPa)
			1	2	3	4	Average		
Dry	3.0	2	39.24	39.36	39.30	39.23	39.28	3.31	357.6
		4	39.45	39.33	39.24	39.36	39.35	3.18	343.0
	3.9	3	39.64	39.68	39.79	39.56	39.67	2.39	255.7
		5	39.64	39.45	39.30	39.38	39.44	2.51	270.1
	4.8	1	41.85	42.08	41.71	41.59	41.81	2.04	207.1
		3	39.15	59.87	39.49	39.13	44.41	2.09	199.7
Soaked	3.0	1	39.97	39.72	39.95	39.31	39.74	3.73	398.4
		3	39.82	39.47	39.58	39.46	39.58	2.98	319.5
	3.9	2	39.63	39.23	39.45	39.67	39.50	2.60	279.4
		4	35.65	35.61	36.38	35.51	35.79	2.36	279.9
	4.8	2	39.92	39.71	39.62	40.16	39.85	1.70	181.0
		4	39.10	39.12	39.14	38.88	39.06	1.98	215.1

C-60L-LE Mixture

Conditioning	Residual Binder Content (%)	Sample	Height (mm)					Peak Load (kN)	Tensile Strength, S_t (kPa)
			1	2	3	4	Average		
Dry	3.0	4	42.74	42.54	42.38	42.21	42.47	3.47	346.8
		5	41.04	40.68	40.77	40.76	40.81	3.4	353.6
	3.9	2	41.34	41.42	41.50	41.37	41.41	2.45	251.1
		3	41.19	41.34	41.09	41.01	41.16	2.38	245.4
	4.8	3	41.12	41.26	41.30	41.28	41.24	2.74	282.0
		5	40.70	41.05	40.96	40.63	40.84	2.06	214.1
Soaked	3.0	2	41.72	41.77	41.52	41.63	41.66	3.53	359.6
		3	41.86	41.66	41.68	41.63	41.71	4.40	447.7
	3.9	1	40.57	40.74	41.00	41.12	40.86	2.47	256.6
		4	41.06	41.15	40.88	40.98	41.02	2.19	226.6
	4.8	1	40.85	40.43	40.10	40.35	40.43	3.34	350.6
		5	40.80	40.64	40.53	40.93	40.73	2.27	236.6

C-60G-GE Mixture

Specimen Conditioning	Residual Binder Content (%)	Sample	Height (mm)					Peak Load (kN)	Tensile Strength, S_t (kPa)
			1	2	3	4	Average		
Dry	2.0	2	43.57	43.14	43.42	43.52	43.41	2.56	250.3
		3	43.14	43.13	42.98	43.03	43.07	2.99	294.6
	3.0	3	41.67	41.42	41.51	41.87	41.62	3.94	401.8
		4	41.46	41.80	41.75	41.72	41.68	3.91	398.1
	4.0	3	41.87	42.00	41.95	41.70	41.88	3.9	395.2
		4	42.36	42.48	42.64	42.37	42.46	3.92	391.8
Soaked	2.0	1	43.62	43.06	43.03	43.18	43.22	2.47	242.5
	3.0	1	42.08	41.53	41.31	41.67	41.65	3.62	368.9
		2	41.80	41.53	41.31	41.67	41.58	3.41	348.1
	4.0	1	41.78	41.83	41.59	41.63	41.71	3.99	406.0
		2	40.87	40.62	40.82	40.98	40.82	3.80	395.1

C. Cold Recycling with Foamed Asphalt

Trial Mixtures ABC-60L-LF Aggregate blend, P_b= 5%, No MC

- Determination of Maximum Specific Gravity (Gmm)

Sample	$W_{mix\text{-}loose}$ (g)	$W_{pyc\ (soak)}$ (g)	$W_{spyc+mix\ (soak)}$ (g)	G_{mm} (-)
1	2288.4	1512.9	2826.5	2.348

- Determination Bulk Specific Gravity (Gsb)

P_b (%)	5.0

Sample	Height (mm)					W_{Dry} (g)	W_{soak} (g)	W_{SSD} (g)	G_{mb} (-)	G_{mm} (-)	AV%
	1	2	3	4	Average						
1	69.92	69.97	70.38	69.87	70.04	1099.50	612.00	1139.00	2.086	2.348	11.1
2	65.78	70.78	70.59	67.47	68.66	1054.80	580.70	1089.50	2.073	2.348	11.7
3	68.42	69.02	68.75	68.21	68.60	1102.20	609.30	1129.00	2.121	2.348	9.7
4	67.94	68.1	68.37	68.55	68.24	1068.40	598.00	1104.50	2.109	2.348	10.2
								Average	2.097		

C-60L-GF Mixture

- Determination of Maximum Specific Gravity (Gmm)

P_b (%)	$W_{mix\text{-}loose}$ (g)	$W_{pyc\ (soak)}$ (g)	$W_{spyc+mix\ (soak)}$ (g)	G_{mm} (-)
2.0	2126.5	1488.4	2753.1	2.468
4.0	2154.1	1488.4	2743.2	2.395
6.0	2166.8	1488.4	2727.3	2.335

- Determination of Bulk Specific Gravity (Gsb)

C-60L-GF	
P_b (%)	2.0

Sample	Height (mm)					W_{Dry} (g)	W_{soak} (g)	W_{SSD} (g)	G_{mb} (-)	G_{mm} (-)	AV%
	1	2	3	4	Average						
1	69.53	69.36	68.01	68.87	68.94	1066.10	563.10	1079.10	2.066	2.468	16.3
2	70.12	70.21	70.29	70.29	70.23	1074.10	567.90	1088.40	2.064	2.468	16.4
3	70.35	70.31	70.64	70.36	70.42	1075.30	565.90	1093.20	2.039	2.468	17.4
4	70.14	70.42	71.11	69.17	70.21	1094.90	583.80	1112.40	2.071	2.468	16.1
								Average	2.060		

C-60L-GF	
P_b (%)	4.0

Sample	Height (mm)					W_{Dry} (g)	W_{soak} (g)	W_{SSD} (g)	G_{mb} (-)	G_{mm} (-)	AV%
	1	2	3	4	Average						
1	70.2	70.42	70.33	70.32	70.32	1080.00	562.50	1094.40	2.030	2.395	15.3
2	70.79	70.79	70.44	70.28	70.58	1087.70	567.20	1101.80	2.035	2.395	15.0
3	70.223	69.78	70.13	70.34	70.12	1083.00	565.90	1097.20	2.038	2.395	14.9
4	69.84	69.42	69.53	69.46	69.56	1075.80	563.70	1090.30	2.043	2.395	14.7
								Average	2.037		

C-60L-GF	
P_b (%)	6.0

Sample	Height (mm)					W_{Dry} (g)	W_{soak} (g)	W_{SSD} (g)	G_{mb} (-)	G_{mm} (-)	AV%
	1	2	3	4	Average						
1	69.62	69.78	70.9	69.43	69.93	1085.50	567.60	1096.80	2.051	2.335	12.2
2	70.04	70.41	70.25	70.05	70.19	1084.60	568.40	1098.60	2.046	2.335	12.4
3	70.36	70.31	70.58	70.59	70.46	1100.10	572.10	1110.80	2.042	2.335	12.6
4	70.41	70.42	69.99	71.04	70.47	1095.20	575.00	1108.20	2.054	2.335	12.0
								Average	2.048		

C-60L-LF Mixture

- Determination of Maximum Specific Gravity (Gmm)

P_b (%)	$W_{mix\text{-}loose}$ (g)	$W_{pyc\ (soak)}$ (g)	$W_{spyc+mix\ (soak)}$ (g)	G_{mm} (-)
2.0	2126	1512.5	2761.2	2.423
4.0	2063.6	1512.5	2700.3	2.356
6.0	2187	1512.5	2740	2.279

- Determination of Bulk Specific Gravity (Gsb)

C-60L-LF	
P_b (%)	2.0

Sample	Height (mm)					W_{Dry} (g)	W_{soak} (g)	W_{SSD} (g)	G_{mb} (-)	G_{mm} (-)	AV%
	1	2	3	4	Average						
1	71.35	71.48	71.41	71.14	71.35	1082.00	556.10	1096.50	2.002	2.423	17.4
2	70.17	71.05	71.37	71.23	70.96	1075.70	554.90	1093.40	1.998	2.423	17.6
3	71.51	71.63	71.55	71.67	71.59	1080.30	558.40	1102.90	1.984	2.423	18.1
4	69.64	71.33	71.42	71.15	70.89	1067.70	553.00	1088.10	1.995	2.423	17.7
								Average	1.995		

C-60L-LF	
P_b (%)	4.0

Sample	Height (mm)					W_{Dry} (g)	W_{soak} (g)	W_{SSD} (g)	G_{mb} (-)	G_{mm} (-)	AV%
	1	2	3	4	Average						
1	71.76	71.75	71.16	70.92	71.40	1067.80	545.50	1082.60	1.988	2.356	15.6
2	72	70.83	70.7	71.16	71.17	1055.40	543.30	1072.70	1.994	2.356	15.4
3	71.22	71.91	72.49	72.22	71.96	1077.30	547.90	1089.60	1.989	2.356	15.6
4	73.33	73.03	72.67	71.61	72.66	1086.50	556.80	1103.50	1.987	2.356	15.7
								Average	1.990		

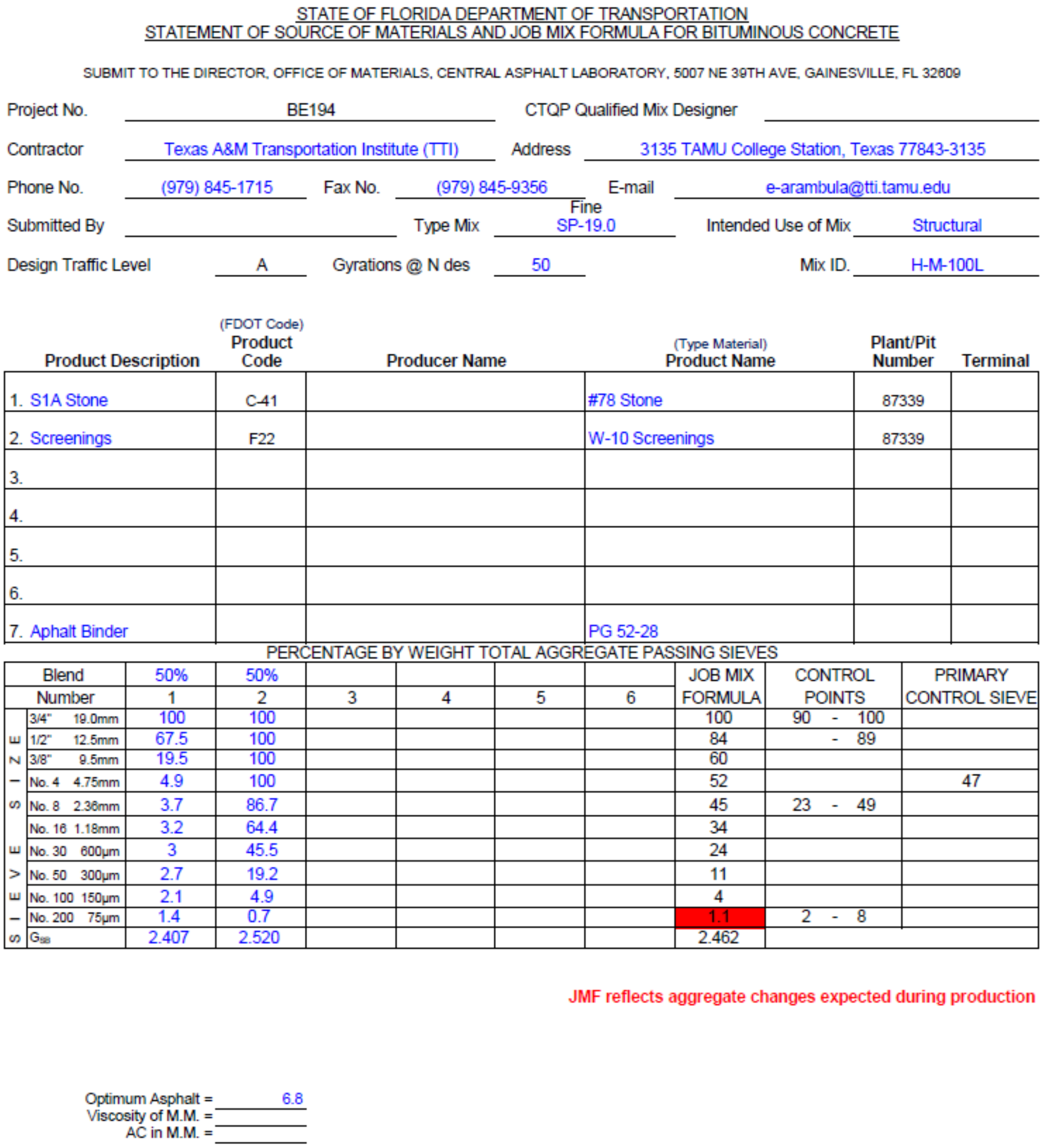
C-60L-LF	
P_b (%)	**6.0**

Sample	**Height (mm)**					**W_{Dry} (g)**	**W_{soak} (g)**	**W_{SSD} (g)**	**G_{mb} (-)**	**G_{mm} (-)**	**AV%**
	1	**2**	**3**	**4**	**Average**						
1	72.95	73.13	72.37	72.39	72.71	1105.50	556.60	1113.20	1.986	2.279	12.9
2	72.76	72.55	72.62	72.8	72.68	1100.70	555.50	1109.60	1.986	2.279	12.9
3	71.94	71.48	71.66	72.02	71.78	1095.50	552.70	1102.80	1.991	2.279	12.6
4	72.25	72.41	72.45	71.91	72.26	1090.50	551.70	1102.00	1.982	2.279	13.0
								Average	1.986		

APPENDIX K

HOT MIX DESIGN RESULTS – FDOT FORMAT

A. Limestone Mixture



STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION
STATEMENT OF SOURCE OF MATERIALS AND JOB MIX FORMULA FOR BITUMINOUS CONCRETE

SUBMIT TO THE DIRECTOR, OFFICE OF MATERIALS, CENTRAL ASPHALT LABORATORY, 5007 NE 39TH AVE, GAINESVILLE, FL 32609

Project No. BE194 CTQP Qualified Mix Designer

Contractor Texas A&M Transportation Institute (TTI) Address 3135 TAMU College Station, Texas 77843-3135

Phone No. (979) 845-1715 Fax No. (979) 845-9356 E-mail e-arambula@tti.tamu.edu

Submitted By Type Mix Fine SP-19.0 Intended Use of Mix Structural

Design Traffic Level A Gyrations @ N des 50 Mix ID. H-M-100L

Product Description	(FDOT Code) Product Code	Producer Name	(Type Material) Product Name	Plant/Pit Number	Terminal
1. S1A Stone	C-41		#78 Stone	87339	
2. Screenings	F22		W-10 Screenings	87339	
3.					
4.					
5.					
6.					
7. Aphalt Binder			PG 52-28		

PERCENTAGE BY WEIGHT TOTAL AGGREGATE PASSING SIEVES

	Blend	50%	50%					JOB MIX	CONTROL	PRIMARY
	Number	1	2	3	4	5	6	FORMULA	POINTS	CONTROL SIEVE
SIEVE SIZE	3/4" 19.0mm	100	100					100	90 - 100	
	1/2" 12.5mm	67.5	100					84	- 89	
	3/8" 9.5mm	19.5	100					60		
	No. 4 4.75mm	4.9	100					52		47
	No. 8 2.36mm	3.7	86.7					45	23 - 49	
	No. 16 1.18mm	3.2	64.4					34		
	No. 30 600µm	3	45.5					24		
	No. 50 300µm	2.7	19.2					11		
	No. 100 150µm	2.1	4.9					4		
	No. 200 75µm	1.4	0.7					🟥 1.1	2 - 8	
	G_{SB}	2.407	2.520					2.462		

JMF reflects aggregate changes expected during production

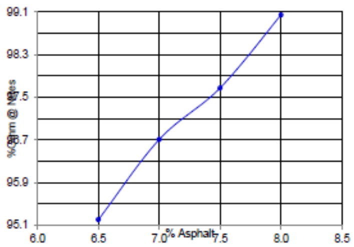
Optimum Asphalt = 6.8
Viscosity of M.M. =
AC in M.M. =

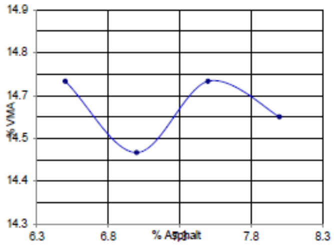
Rev. 01/12/2007

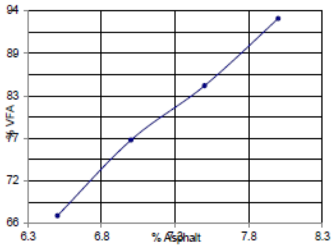
HOT MIX DESIGN DATA SHEET

H-M-100L

P_b	G_{mb} @ N_{des}	G_{mm}	V_a	VMA	VFA	P_{be}	$P_{0.075}$ / P_{be}	%G_{mm} @ N_{ini}	%G_{mm} @ N_{max}
6.5	2.245	2.358	4.8	14.7	67.35	4.6	0.2	89.3	
7.0	2.264	2.341	3.3	14.5	77.24	5.1	0.2	90.5	
7.5	2.270	2.324	2.3	14.7	84.35	5.6	0.2	91.4	
8.0	2.286	2.308	1.0	14.6	93.15	6.1	0.2	92.3	







Total Binder Content 6.8 %

FAA ______ %

Mixing Temperature 275 °F 135 °C

Spread Rate @ 1" 102 lbs/yd^2

%G_{mm} @ N_{des} 95.2

Compaction Temperature 275 °F 135 °C

VMA 14.7 %

Ignition Oven Calibration Factor ______
(+To Be Added)/(-To Be Subtracted)

Additives Antistrip 0.5 % ______ %

G_{mm} Corr. Factor ______

B. Granite Mixture

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION
STATEMENT OF SOURCE OF MATERIALS AND JOB MIX FORMULA FOR BITUMINOUS CONCRETE

SUBMIT TO THE DIRECTOR, OFFICE OF MATERIALS, CENTRAL ASPHALT LABORATORY, 5007 NE 39TH AVE, GAINESVILLE, FL 32609

Project No. BE194 CTQP Qualified Mix Designer

Contractor Texas A&M Transportation Institute (TTI) Address 3135 TAMU College Station, Texas 77843-3135

Phone No. (979) 845-1715 Fax No. (979) 845-9356 E-mail e-arambula@tti.tamu.edu

Submitted By Type Mix Fine SP-12.5 Intended Use of Mix Structural

Design Traffic Level A Gyrations @ N des 50 Mix ID. H-M-100G

Product Description	(FDOT Code) Product Code	Producer Name	(Type Material) Product Name	Plant/Pit Number	Terminal
1. S1A Stone	C-47		#78 Stone	GA-553	
2. Screenings	F22		W-10 Screenings	GA-553	
3.					
4.					
5.					
6.					
7. Aphalt Binder			PG 52-28		

PERCENTAGE BY WEIGHT TOTAL AGGREGATE PASSING SIEVES

SIEVE SIZE	Blend	40%	60%					JOB MIX FORMULA	CONTROL POINTS	PRIMARY CONTROL SIEVE
	Number	1	2	3	4	5	6			
	3/4" 19.0mm	100	100					100	100	
	1/2" 12.5mm	93.3	100					97	90 - 100	
	3/8" 9.5mm	49.4	100					80	- 89	
	No. 4 4.75mm	11.3	97.2					63		
	No. 8 2.36mm	3.7	72					45	28 - 58	39
	No. 16 1.18mm	2	47.3					29		
	No. 30 600μm	1.5	31.2					19		
	No. 50 300μm	1.3	19.7					12		
	No. 100 150μm	1.1	9.9					6		
	No. 200 75μm	0.7	4.1					2.7	2 - 10	
	G_{SB}	2.775	2.740					2.754		

JMF reflects aggregate changes expected during production

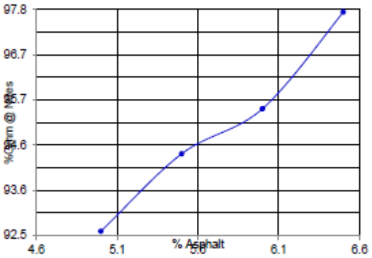
Optimum Asphalt = 6.0
Viscosity of M.M. =
AC in M.M. =

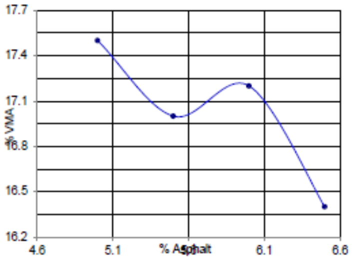
Rev. 01/12/2007

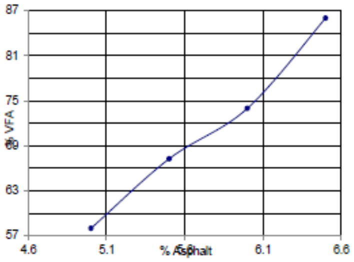
HOT MIX DESIGN DATA SHEET

H-M-100G

P_b	G_{mb} @ N_{des}	G_{mm}	V_a	VMA	VFA	P_{be}	$P_{0.075}$ / P_{be}	%G_{mm} @ N_{ini}	%G_{mm} @ N_{max}
5.0	2.392	2.583	7.4	17.5	57.71	4.3	0.6	85.6	
5.5	2.419	2.562	5.6	17.0	67.06	4.9	0.6	87.0	
6.0	2.426	2.541	4.5	17.2	73.84	5.4	0.5	87.7	
6.5	2.463	2.520	2.3	16.4	85.98	5.9	0.5	89.7	







Total Binder Content 6.0 %

FAA ________ %

Mixing Temperature 275 °F 135 °C

Spread Rate @ 1" 110 lbs/yd^2

%G_{mm} @ N_{des} 95.5

Compaction Temperature 275 °F 135 °C

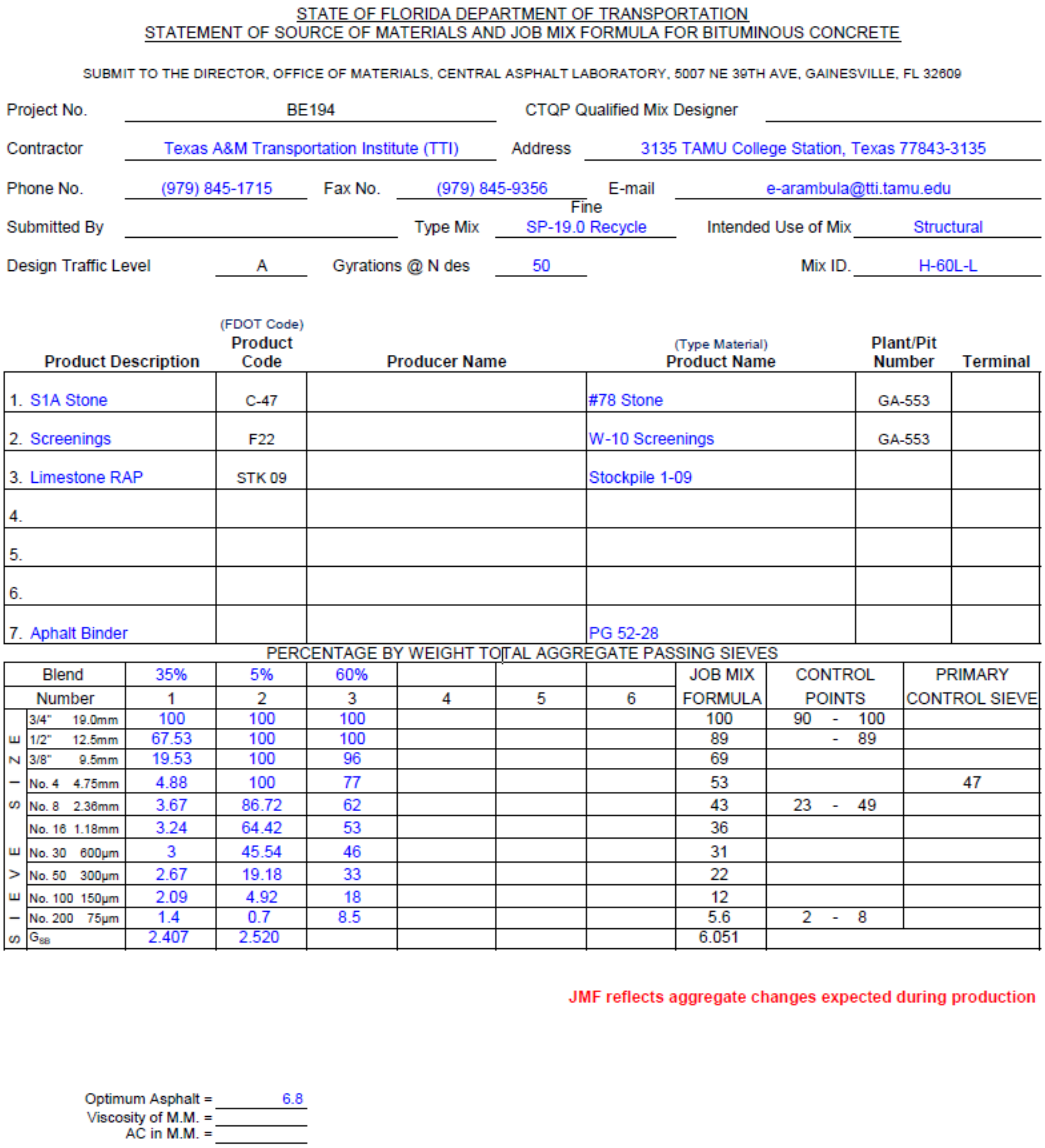
VMA 17.2 %

Ignition Oven Calibration Factor ________
(+To Be Added)/(-To Be Subtracted)

Additives Antistrip 0.5 % ________ %

G_{mm} Corr. Factor ________

C. H-60L-L



STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION
STATEMENT OF SOURCE OF MATERIALS AND JOB MIX FORMULA FOR BITUMINOUS CONCRETE

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Project No. BE194 CTQP Qualified Mix Designer

Contractor Texas A&M Transportation Institute (TTI) Address 3135 TAMU College Station, Texas 77843-3135

Phone No. (979) 845-1715 Fax No. (979) 845-9356 E-mail e-arambula@tti.tamu.edu

Submitted By Type Mix Fine SP-19.0 Recycle Intended Use of Mix Structural

Design Traffic Level A Gyrations @ N des 50 Mix ID. H-60L-L

Product Description	(FDOT Code) Product Code	Producer Name	(Type Material) Product Name	Plant/Pit Number	Terminal
1. S1A Stone	C-47		#78 Stone	GA-553	
2. Screenings	F22		W-10 Screenings	GA-553	
3. Limestone RAP	STK 09		Stockpile 1-09		
4.					
5.					
6.					
7. Aphalt Binder			PG 52-28		

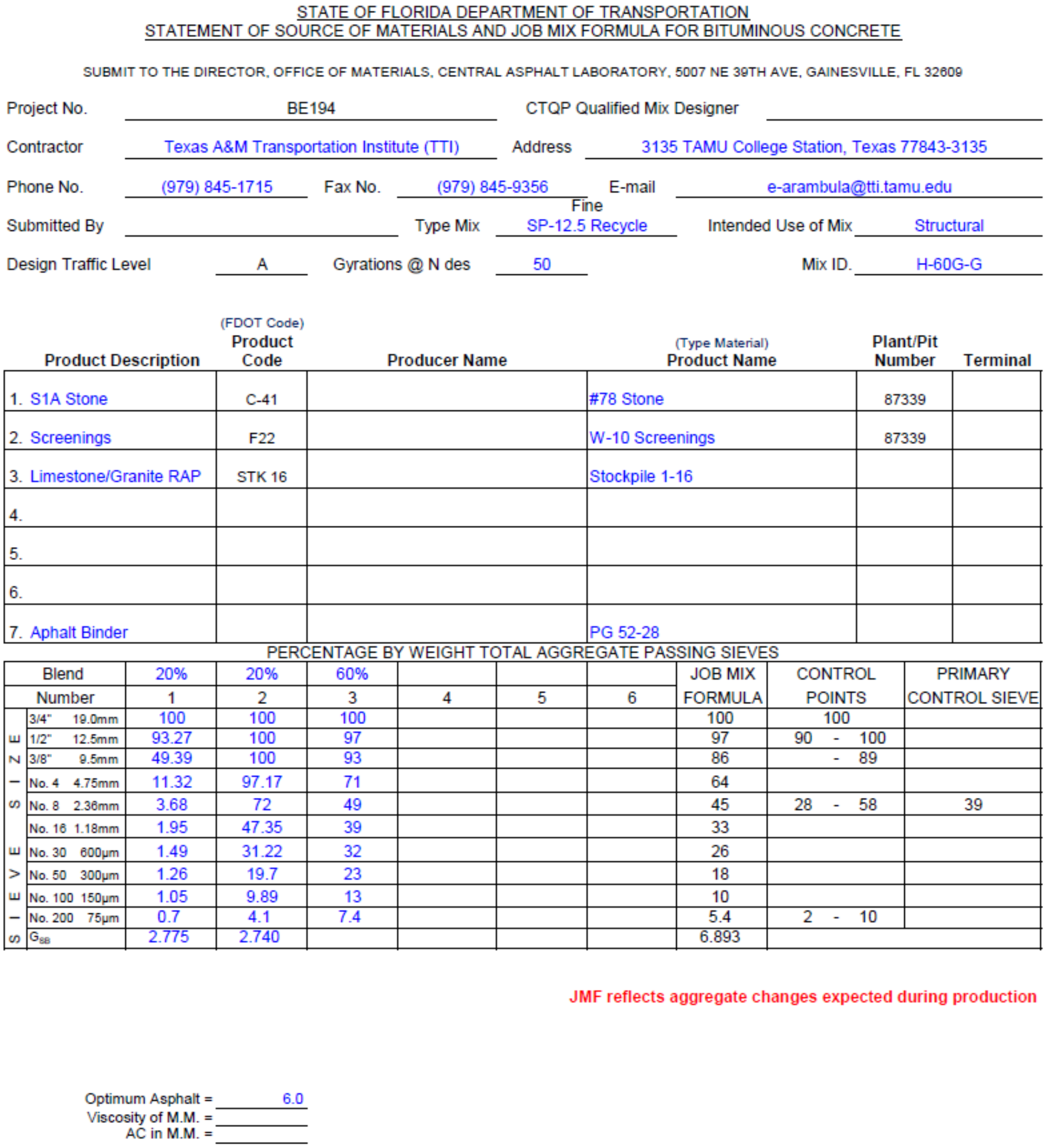
PERCENTAGE BY WEIGHT TOTAL AGGREGATE PASSING SIEVES

	Blend	35%	5%	60%				JOB MIX	CONTROL	PRIMARY
	Number	1	2	3	4	5	6	FORMULA	POINTS	CONTROL SIEVE
SIEVE SIZE	3/4" 19.0mm	100	100	100				100	90 - 100	
	1/2" 12.5mm	67.53	100	100				89	- 89	
	3/8" 9.5mm	19.53	100	96				69		
	No. 4 4.75mm	4.88	100	77				53		47
	No. 8 2.36mm	3.67	86.72	62				43	23 - 49	
	No. 16 1.18mm	3.24	64.42	53				36		
	No. 30 600μm	3	45.54	46				31		
	No. 50 300μm	2.67	19.18	33				22		
	No. 100 150μm	2.09	4.92	18				12		
	No. 200 75μm	1.4	0.7	8.5				5.6	2 - 8	
	G_{SB}	2.407	2.520					6.051		

JMF reflects aggregate changes expected during production

Optimum Asphalt = 6.8
Viscosity of M.M. =
AC in M.M. =

D. H-60G-G



STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION
STATEMENT OF SOURCE OF MATERIALS AND JOB MIX FORMULA FOR BITUMINOUS CONCRETE

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CTQP Qualified Mix Designer
Contractor Texas A&M Transportation Institute (TTI)
Address 3135 TAMU College Station, Texas 77843-3135
Phone No. (979) 845-1715
Fax No. (979) 845-9356
E-mail e-arambula@tti.tamu.edu
Submitted By
Type Mix Fine SP-12.5 Recycle
Intended Use of Mix Structural
Design Traffic Level A
Gyrations @ N des 50
Mix ID. H-60G-G

Product Description	(FDOT Code) Product Code	Producer Name	(Type Material) Product Name	Plant/Pit Number	Terminal
1. S1A Stone	C-41		#78 Stone	87339	
2. Screenings	F22		W-10 Screenings	87339	
3. Limestone/Granite RAP	STK 16		Stockpile 1-16		
4.					
5.					
6.					
7. Aphalt Binder			PG 52-28		

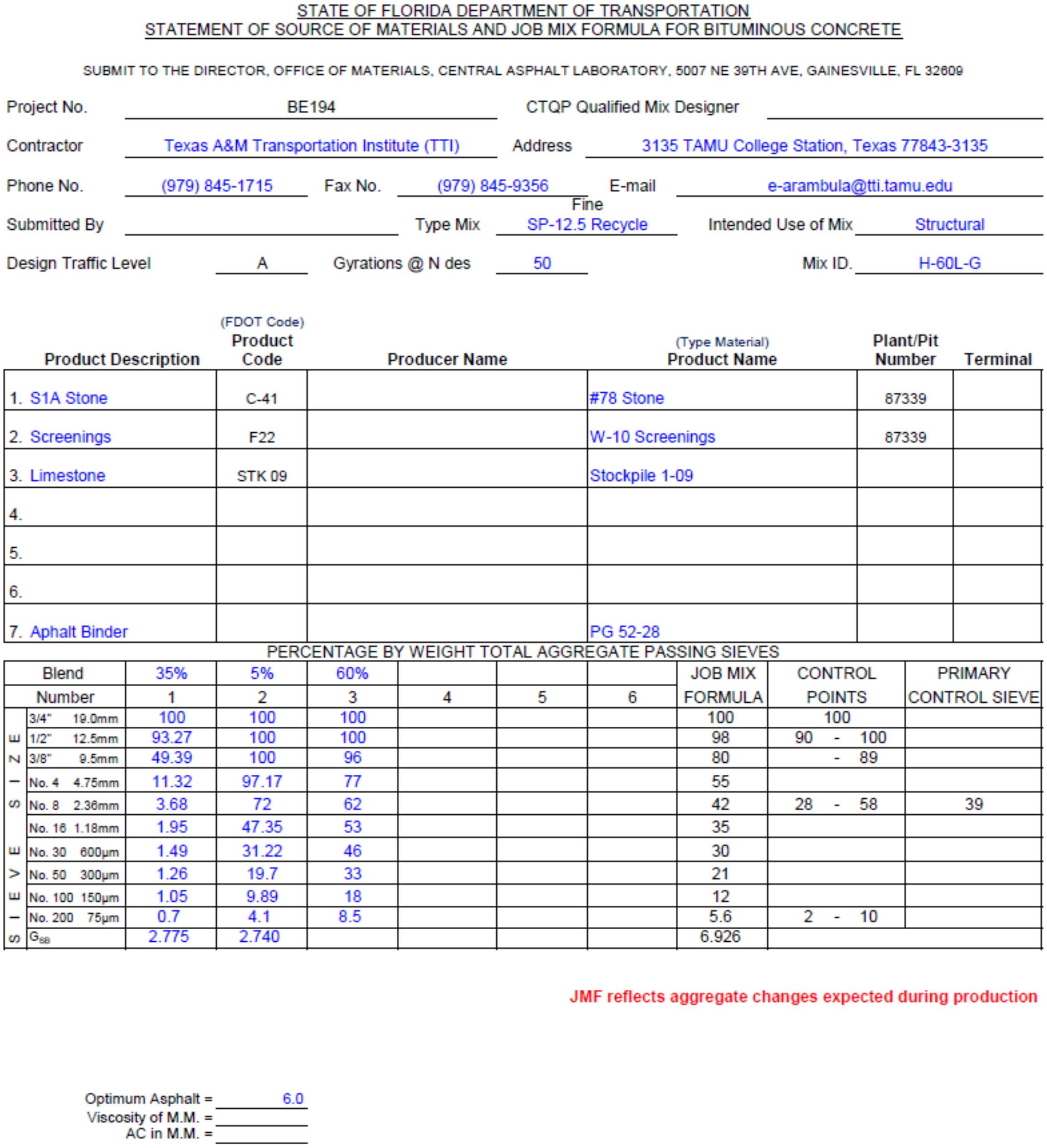
PERCENTAGE BY WEIGHT TOTAL AGGREGATE PASSING SIEVES

	Blend	20%	20%	60%				JOB MIX	CONTROL	PRIMARY
	Number	1	2	3	4	5	6	FORMULA	POINTS	CONTROL SIEVE
SIEVE SIZE	3/4" 19.0mm	100	100	100				100	100	
	1/2" 12.5mm	93.27	100	97				97	90 - 100	
	3/8" 9.5mm	49.39	100	93				86	- 89	
	No. 4 4.75mm	11.32	97.17	71				64		
	No. 8 2.36mm	3.68	72	49				45	28 - 58	39
	No. 16 1.18mm	1.95	47.35	39				33		
	No. 30 600µm	1.49	31.22	32				26		
	No. 50 300µm	1.26	19.7	23				18		
	No. 100 150µm	1.05	9.89	13				10		
	No. 200 75µm	0.7	4.1	7.4				5.4	2 - 10	
	G_{SB}	2.775	2.740					6.893		

JMF reflects aggregate changes expected during production

Optimum Asphalt = 6.0
Viscosity of M.M. =
AC in M.M. =

E. H-60L-G



STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION
STATEMENT OF SOURCE OF MATERIALS AND JOB MIX FORMULA FOR BITUMINOUS CONCRETE

SUBMIT TO THE DIRECTOR, OFFICE OF MATERIALS, CENTRAL ASPHALT LABORATORY, 5007 NE 39TH AVE, GAINESVILLE, FL 32609

Project No. BE194
CTQP Qualified Mix Designer
Contractor Texas A&M Transportation Institute (TTI)
Address 3135 TAMU College Station, Texas 77843-3135
Phone No. (979) 845-1715
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Submitted By
Type Mix Fine SP-12.5 Recycle
Intended Use of Mix Structural
Design Traffic Level A
Gyrations @ N des 50
Mix ID. H-60L-G

Product Description	(FDOT Code) Product Code	Producer Name	(Type Material) Product Name	Plant/Pit Number	Terminal
1. S1A Stone	C-41		#78 Stone	87339	
2. Screenings	F22		W-10 Screenings	87339	
3. Limestone	STK 09		Stockpile 1-09		
4.					
5.					
6.					
7. Aphalt Binder			PG 52-28		

PERCENTAGE BY WEIGHT TOTAL AGGREGATE PASSING SIEVES

	Blend	35%	5%	60%				JOB MIX	CONTROL	PRIMARY
	Number	1	2	3	4	5	6	FORMULA	POINTS	CONTROL SIEVE
SIEVE SIZE	3/4" 19.0mm	100	100	100				100	100	
	1/2" 12.5mm	93.27	100	100				98	90 - 100	
	3/8" 9.5mm	49.39	100	96				80	- 89	
	No. 4 4.75mm	11.32	97.17	77				55		
	No. 8 2.36mm	3.68	72	62				42	28 - 58	39
	No. 16 1.18mm	1.95	47.35	53				35		
	No. 30 600μm	1.49	31.22	46				30		
	No. 50 300μm	1.26	19.7	33				21		
	No. 100 150μm	1.05	9.89	18				12		
	No. 200 75μm	0.7	4.1	8.5				5.6	2 - 10	
	G_{SB}	2.775	2.740					6.926		

JMF reflects aggregate changes expected during production

Optimum Asphalt = 6.0
Viscosity of M.M. =
AC in M.M. =

APPENDIX L

CURING PROTOCOL EXPERIMENT

A. C-100L-E Mixtures

EC (RBC) (%)	**6.5 (3.9)**							
Curing Time (Hr)		**1.0**	**3.0**	**5.0**	**7.0**	**19.0**	**21.0**	**25.0**
Sample	1	1334.5	1328.5	1325.5	1323.5	1313.5	1312.0	1311.5
	2	1349.0	1341.5	1338.0	1336.0	1327.5	1327.0	1326.5
	3	1350.5	1343.0	1338.5	1335.0	1326.5	1323.5	1323.0
	4	1326.0	1317.0	1313.5	1311.0	1302.5	1301.5	1300.5

		Weight Change (%)					
Curing Time (Hr)		**3.0**	**5.0**	**7.0**	**19.0**	**21.0**	**25.0**
Sample	1	0.45	0.23	0.15	NA	0.11	0.04
	2	0.56	0.26	0.15	NA	0.04	0.04
	3	0.56	0.34	0.26	NA	0.23	0.04
	4	0.68	0.27	0.19	NA	0.08	0.08
	Average	0.56	0.27	0.19	NA	0.11	0.05

EC (RBC) (%)	**8.0 (4.8)**							
Curing Time (Hr)		**2.0**	**4.0**	**6.0**	**8.0**	**20.0**	**22.0**	**26.0**
Sample	1	1447.0	1443.0	1441.0	1440.0	1434.0	1433.5	1433.0
	2	1361.0	1358.0	1356.5	1355.5	1350.0	1349.0	1348.5
	3	1349.0	1346.5	1344.5	1343.5	1336.5	1335.5	1334.5
	4	1227.5	1224.0	1222.0	1221.0	1214.5	1214.0	1213.5

		Weight Change (%)					
Curing Time (Hr)		**4.0**	**6.0**	**8.0**	**20.0**	**22.0**	**26.0**
Sample	1	0.28	0.14	0.07	NA	0.03	0.03
	2	0.22	0.11	0.07	NA	0.07	0.04
	3	0.19	0.15	0.07	NA	0.07	0.07
	4	0.29	0.16	0.08	NA	0.04	0.04
	Average	0.24	0.14	0.07	NA	0.06	0.05

B. C-60L-LE Mixture

EC (RBC) (%)	**6.5 (3.9)**

Curing Time (Hr)		**0.5**	**2.0**	**4.0**	**6.0**	**18.0**	**20.0**	**24.0**	**26.0**
Sample	1	1445.5	1437.0	1433.0	1430.5	1420.5	1420.0	1419.5	1419.0
	2	1449.5	1437.0	1433.0	1430.0	1420.5	1420.0	1419.5	1419.0
	3	1462.5	1451.5	1446.5	1444.0	1434.5	1433.5	1433.0	1432.5
	4	1458.5	1448.5	1445.0	1442.5	1433.0	1432.5	1432.0	1431.5

Curing Time (Hr)		**2.0**	**4.0**	**6.0**	**18.0**	**20.0**	**24.0**	**26.0**
Sample	1	0.59	0.28	0.17	NA	0.04	0.04	0.04
	2	0.86	0.28	0.21	NA	0.04	0.04	0.04
	3	0.75	0.34	0.17	NA	0.07	0.03	0.03
	4	0.69	0.24	0.17	NA	0.03	0.03	0.03
	Average	0.72	0.29	0.18	NA	0.04	0.04	0.04

APPENDIX M

EXPANSION RATIO (ER) AND HALF LIFE (H-L) TESTS

A. Foaming Temperature Selection

1-Gallon Can Diameter	16.5 cm	6.5 in
Mass of Dispensed Asphalt	200.0 g	0.44 lb.
Measured Thickness of Un-foamed Asphalt in 1-Gallon Can	0.91 cm	0.36 in
Volume of Un-foamed Asphalt (ft^3)	0.000195	11.9 in3

Foaming Temperature (°C)	**Water Injection Rate (WIR), (%)**	**Measurement of Foam Height on the Can Wall (cm)**						**Asphalt Layer Thickness in the Can base (cm)**			
		#1	**#2**	**#3**	**#4**	**#5**	**Average**	**#1**	**#2**	**#3**	**Average**
160	1.0	9.9	9.6				9.8	0.9	0.8		0.9
	2.0	12.9	13.9				13.4	1.1	0.9		1.0
	3.0	12.8	13.2	13.4			13.1	0.8	1.0		0.9
	5.0	15.7	16.2	16.4			16.1	0.8	0.8		0.8
170	1.0	7.8	7.9	8.1	8.8	9.0	8.3	1.0	1.0		1.0
	2.0	12.5	13.0	13.3			12.9	1.0	1.0	0.9	1.0
	3.0	13.5	13.7	13.9	14.3		13.9	0.9	0.9		0.9
	5.0	14.8	15.0				14.9	0.9	0.9		0.9

Foaming Temperature (°C)	**Maximum Foaming Height (cm)**	**Max ER (Times)**	**1/2 Max ER (-)**	**ER Fitting Constants**			**$1/2ER_{Max}(HL) = 1 + (a \cdot e^{-b \cdot HL} + (ER_{Max} - a - 1)\, e^{-c \cdot HL})$**	**HL (Sec)**
				a	**b**	**c**		
160	10.60	11.65	5.82	7.14	0.66	0.01	5.81	2.44
	14.40	15.82	7.91	11.85	0.40	0.01	7.89	2.72
	14.03	15.42	7.71	9.39	0.67	0.05	7.69	2.20
	16.88	18.54	9.27	7.08	8.32	0.07	9.26	3.30
170	9.32	10.24	5.12	6.28	0.33	0.01	5.11	4.67
	13.90	15.27	7.64	11.19	0.34	0.02	7.62	3.26
	14.75	16.21	8.10	10.40	0.94	0.04	8.09	1.49
	15.75	17.31	8.65	7.40	11.95	0.07	8.64	2.32

B. Optimum Water Injection Rate (OWIR) Determination

WIR (%)	Rep.	Initial Mass (g)	Final Mass (g)	Binder Mass (g)	Volume (m³)	Height of Layer (cm)	Average Layer Height of Un-foamed Binder (cm)
0.7	1	287.2	488.3	201.1	1.92E-04	0.89	0.90
	2	282.3	485.5	203.2	1.94E-04	0.90	
1.5	1	282.4	485.9	203.5	1.94E-04	0.91	0.90
	2	285.5	488.0	202.5	1.93E-04	0.90	
3.0	1	284.4	491.1	206.7	1.97E-04	0.92	0.91
	2	279.3	482.0	202.7	1.93E-04	0.90	
4.0	1	282.9	492.1	209.2	1.99E-04	0.93	0.93
	2	283.6	492.1	208.5	1.99E-04	0.93	

WIR (%)	Rep.	Foaming Height (cm)					Maximum Expansion Height (cm)
		#1	#2	#3	#4	Average	
0.7	1	12.7	12.8	12.5		12.7	5.5
	2	12.5	12.9	12.3		12.6	5.6
1.5	1	5.9	5.0	5.4		5.4	12.8
	2	8.1	8.0	7.8		8.0	10.2
3.0	1	3.5	3.6	3.7		3.6	14.6
	2	3.6	3.1	3.3		3.3	14.9
4.0	1	2.7	1.8	2.5	1.8	2.2	16.0
	2	1.7	1.5	2.1	2.2	1.9	16.3

WIR (%)	Rep.	ER Fitting Constants			Max ER (Times)	1/2 Max ER (-)	HL (Sec)
		a	b	c			
0.7	1	2.817	0.005	14.100	6.2	3.11	61.5
	2	2.179	0.183	0.004	6.3	3.17	94.5
1.5	1	10.226	0.256	0.006	14.3	7.17	4.6
	2	7.048	1.451	0.006	11.5	5.75	1.7
3.0	1	11.801	0.337	0.009	16.4	8.20	3.4
	2	12.338	0.371	0.007	16.7	8.35	3.0
4.0	1	14.130	0.226	0.009	18.0	8.99	4.4
	2	13.878	0.373	0.010	18.3	9.17	2.9

APPENDIX N

VOLUMETRICS OF PERFORMANCE TESTING SPECIMENS

A. Hot Recycling

Determination of Maximum Specific Gravity (G_{mm})

Mix ID	$W_{mix\text{-}loose}$ (g)	$W_{pyc\ (soak)}$ (g)	$W_{spyc+mix\ (soak)}$ (g)	G_{mm} (-)	G_{mm} (-)
H-60L-L	1816.9	1512.8	2549.9	2.330	2.343
	1849.9	1489.6	2554.1	2.355	
H-60L-G	1375.9	1512.8	2328.8	2.457	2.460
	1380.1	1489.6	2309.3	2.463	
H-60G-G	1834.1	1512.8	2610.4	2.490	2.493
	1859.8	1489.6	2604.2	2.496	

Determination of Bulk Specific Gravity (G_{sb})

- Moisture Susceptibility Specimens

Mix ID	Specimen	Height (mm)					W_{Dry} (g)	W_{soak} (g)	W_{SSD} (g)	G_{mb} (-)	G_{mm} (-)	AV (%)
		1	2	3	4	Average						
H-60G-G	1	38.96	38.26	38.55	39.43	38.80	1541.4	886.1	1551.2	2.318	2.493	7.0
	2	36.18	36.34	36.62	35.78	36.23	1419.3	814.3	1432.5	2.296	2.493	7.9
	3	39.56	39.9	39.32	38.83	39.40	1559.3	897.7	1572.9	2.309	2.493	7.4
	4	35.53	35.7	35.26	35.02	35.38	1398.3	807.5	1411.4	2.315	2.493	7.1
	5	36.05	36.94	37.89	36.8	36.92	1463.2	839.6	1472.3	2.313	2.493	7.2
	6	37.29	39.07	38.69	36.93	38.00	1494.3	857.7	1504.3	2.311	2.493	7.3
	7	38.22	37.4	38.17	39.63	38.36	1530	878.1	1537.9	2.319	2.493	7.0
	8	35.78	36.29	37.58	37.56	36.80	1423.6	820.9	1437.1	2.310	2.493	7.3

- Moisture Susceptibility Specimens (continued)

Mix ID	**Specimen**	**Height (mm)**					**W_{Dry} (g)**	**W_{soak} (g)**	**W_{SSD} (g)**	**G_{mb} (-)**	**G_{mm} (-)**	**AV (%)**
		1	**2**	**3**	**4**	**Average**						
H-60G-GO	1	35.98	36.9	37.48	36.48	36.71	1456.5	838.9	1469.5	2.310	2.493	7.3
	2	37.52	37.65	38.96	39.29	38.36	1502.5	865.9	1512.4	2.324	2.493	6.8
	3	36.34	37.44	37.44	35.82	36.76	1454	835	1466.9	2.301	2.493	7.7
	4	38.46	37.37	38.12	39.15	38.28	1507.3	864.4	1518.4	2.305	2.493	7.5
	5	36.55	36.79	38.37	38.35	37.52	1475.5	843	1486.1	2.294	2.493	8.0
	6	37.01	36.85	38.26	38.37	37.62	1479.8	847	1492.4	2.293	2.493	8.0
	7	37.35	38.7	39.89	38.32	38.57	1528.1	874	1539	2.298	2.493	7.8
	8	35.92	37.89	37.18	34.66	36.41	1432.8	819	1441.9	2.300	2.493	7.7
H-60G-GP	1	37.83	38.2	38.208	37.87	38.03	1501.6	866.9	1515.8	2.314	2.493	7.2
	2	36.81	36.7	36.97	37.41	36.97	1457.9	839.9	1469.7	2.315	2.493	7.1
	3	39.97	37.417	36.89	38.81	38.27	1515.1	866.2	1528.7	2.287	2.493	8.3
	4	36.87	39.46	36.87	34.89	37.02	1435.3	820	1451.7	2.272	2.493	8.9
	5	37.05	36.63	37.19	37.79	37.17	1466.9	841.9	1479.2	2.302	2.493	7.7
	6	37.37	36.8	38.26	38.17	37.65	1486	855.6	1489.6	2.344	2.493	6.0
	7	38.36	40.23	39.56	37.43	38.90	1530.1	880.8	1544.1	2.307	2.493	7.5
	8	37.57	35.54	35.28	36.99	36.35	1428.4	826	1444.3	2.310	2.493	7.3
H-60L-L	1	36.25	36.31	36.93	36.77	36.57	1408.4	782.2	1410.9	2.240	2.343	4.4
	2	37.66	38.04	38.3	38.19	38.05	1406.5	783.5	1416.6	2.222	2.343	5.2
	3	36.69	36.58	37.18	37.36	36.95	1423.1	793.8	1427.1	2.247	2.343	4.1
	4	37.63	36.69	37.09	37.68	37.27	1392.1	779.6	1399.3	2.246	2.343	4.1
	5	36.23	37.41	37.35	35.88	36.72	1401.2	778.6	1404.3	2.239	2.343	4.4
	6	38.44	38.98	37.76	37.43	38.15	1418.8	791.7	1426.2	2.236	2.343	4.6
	7	37.25	38.03	36.65	36.22	37.04	1398.5	781	1406.3	2.237	2.343	4.5
	8	37.88	38.15	36.99	37.19	37.55	1427	799.4	1435.1	2.245	2.343	4.2
H-60L-LO	1	37.04	38.59	39.68	37.86	38.29	1455.4	807.9	1459	2.235	2.343	4.6
	2	36.41	37.56	35.68	35.19	36.21	1356.9	758.2	1363.2	2.243	2.343	4.3
	3	38.81	39.09	38.07	37.63	38.40	1463.5	813.3	1470.7	2.226	2.343	5.0
	4	36.4	36.48	34.86	35.19	35.73	1355.2	756.8	1361	2.243	2.343	4.3
	5	37.22	36.48	35.33	36.08	36.28	1368.1	758.8	1375.6	2.218	2.343	5.3
	6	37.31	38.63	39.71	38.21	38.47	1455.2	808.9	1459.7	2.236	2.343	4.6
	7	38.81	38	37.54	37.96	38.08	1451.2	808.5	1457.9	2.235	2.343	4.6
	8	36.27	36.88	36.56	35.73	36.36	1363.4	763.4	1369.8	2.248	2.343	4.1

- Moisture Susceptibility Specimens (continued)

Mix ID	**Specimen**	**Height (mm)**					**W_{Dry} (g)**	**W_{soak} (g)**	**W_{SSD} (g)**	**G_{mb} (-)**	**G_{mm} (-)**	**AV (%)**
		1	**2**	**3**	**4**	**Average**						
H-60L-LP	1	36.69	35.95	36.18	37.26	36.52	1390.1	773.6	1393.6	2.242	2.343	4.3
	2	37.99	37.36	38	38.89	38.06	1430.4	799.1	1436.7	2.243	2.343	4.3
	3	39.31	38.39	37.6	38.32	38.41	1378.4	767.3	1381.4	2.245	2.343	4.2
	4	34.87	35.34	36.46	35.99	35.67	1443.3	806.1	1448.3	2.247	2.343	4.1
	5	38.47	37.27	37.5	38.11	37.84	1460.9	809.9	1466.5	2.225	2.343	5.0
	6	36.66	36.49	36	36.23	36.35	1361.9	756.3	1368.6	2.224	2.343	5.1
	7	38.76	37.62	37.31	38.8	38.12	1453.9	809.1	1458.9	2.237	2.343	4.5
	8	37.03	37.27	35.85	35.73	36.47	1364.4	762	1369	2.248	2.343	4.1
H-60L-GO	1	36.1	37.47	36.83	35.2	36.40	1401.7	799.7	1414.2	2.281	2.460	7.3
	2	37.78	39.01	39.63	38.15	38.64	1485.6	853.3	1495.2	2.314	2.460	5.9
	3	37.45	37.21	36.01	36.29	36.74	1420.3	807.5	1431	2.278	2.460	7.4
	4	38.9	39.2	38.12	37.77	38.50	1476.9	843.6	1491.5	2.280	2.460	7.3
	5	38.41	39.5	38.49	37.53	38.48	1511	855.3	1519.4	2.275	2.460	7.5
	6	34.76	35.26	36.25	35.92	35.55	1382.5	788.1	1392.4	2.288	2.460	7.0
	7	37.22	37.43	35.84	34.85	36.34	1416.9	806.6	1425.1	2.291	2.460	6.9
	8	37.4	38.12	38.83	37.8	38.04	1470.2	839.9	1482.3	2.289	2.460	7.0
H-60L-GP	1	36.3	35.72	34.96	35.47	35.61	1387.6	789.2	1397.9	2.280	2.460	7.3
	2	38.15	38.64	39.04	39.14	38.74	1506.1	857.8	1518.6	2.279	2.460	7.4
	3	34.56	34.74	36.17	36.06	35.38	1393.4	791.6	1400.2	2.290	2.460	6.9
	4	39.23	37.64	38.06	39.45	38.60	1500.3	855.7	1511.2	2.289	2.460	7.0
	5	37.97	37.76	39.27	39.44	38.61	1510.3	861.2	1523.8	2.279	2.460	7.4
	6	35.03	36.53	36.31	34.88	35.69	1378.2	779	1385.2	2.274	2.460	7.6
	7	35.49	36.66	35.93	34.87	35.74	1399.2	797.8	1409.5	2.287	2.460	7.0
	8	38.69	37.74	38.48	38.91	38.46	1487.7	846.6	1497.9	2.284	2.460	7.2

- Rutting Resistance Specimens

Mix ID	**Specimen**	**W_{Dry} (g)**	**W_{soak} (g)**	**W_{SSD} (g)**	**G_{mb} (-)**	**G_{mm} (-)**	**AV (%)**
H-60G-G	1	2518.3	1442.1	2544.6	2.284	2.493	8.4
	2	2520.4	1447.0	2540.2	2.306	2.493	7.5
	3	2515.5	1442.9	2541.3	2.290	2.493	8.1
	4	2522.2	1446.1	2541.5	2.303	2.493	7.6
H-60G-GO	1	2514.2	1445.9	2536.3	2.306	2.493	7.5
	2	2518.4	1448.2	2540.1	2.306	2.493	7.5
	3	2510.7	1434.7	2529.8	2.293	2.493	8.0
	4	2515.2	1440.5	2538.4	2.291	2.493	8.1

- Rutting Resistance Specimens (continued)

Mix ID	**Specimen**	**W_{Dry} (g)**	**W_{soak} (g)**	**W_{SSD} (g)**	**G_{mb} (-)**	**G_{mm} (-)**	**AV (%)**
H-60G-GP	1	2521.3	1451.0	2545.7	2.303	2.493	7.6
	2	2520.5	1447.0	2541.2	2.304	2.493	7.6
	3	2520.8	1444.0	2542.0	2.296	2.493	7.9
	4	2518.6	1428.3	2535.3	2.275	2.493	8.7
H-60L-L	1	2396.9	1320.5	2407.4	2.205	2.343	5.9
	2	2399.9	1332.1	2411.7	2.223	2.343	5.1
	3	2402.2	1324.0	2412.1	2.208	2.343	5.8
	4	2406.1	1332.5	2425.5	2.201	2.343	6.1
H-60L-LO	1	2398.7	1332.2	2408.9	2.228	2.343	4.9
	2	2401.8	1336.7	2412.0	2.234	2.343	4.7
	3	2405.7	1336.1	2416.1	2.228	2.343	4.9
	4	2384.1	1333.3	2399.8	2.235	2.343	4.6
H-60L-LP	1	2395.6	1334.3	2403.8	2.240	2.343	4.4
	2	2402.1	1344.4	2410.5	2.253	2.343	3.8
	3	2382.8	1326.4	2392.9	2.234	2.343	4.7
	4	2405.4	1346.2	2417.2	2.246	2.343	4.1
H-60L-GO	1	2464.1	1407.5	2481.0	2.295	2.460	6.7
	2	2463.2	1404.3	2483.8	2.282	2.460	7.2
	3	2459.5	1402.2	2477.3	2.288	2.460	7.0
	4	2461.6	1401.7	2484.8	2.273	2.460	7.6
H-60L-GP	1	2462.1	1390.4	2477.8	2.264	2.460	8.0
	2	2461.7	1404.7	2479.0	2.291	2.460	6.9
	3	2460.5	1401.8	2479.4	2.283	2.460	7.2
	4	2462.9	1403.2	2483.6	2.280	2.460	7.3

- Intermediate Temperature Cracking Resistance

Mix ID	**Specimen**	**W_{Dry} (g)**	**W_{soak} (g)**	**W_{SSD} (g)**	**G_{mb} (-)**	**G_{mm} (-)**	**AV (%)**
H-60G-G	1	1984.6	1139.5	2001.6	2.302	2.493	7.7
	2	1984.6	1143.5	2008	2.296	2.493	7.9
	3	1985.9	1139	2002.4	2.300	2.493	7.7
H-60G-GO	1	1986	1142.8	2007.7	2.296	2.493	7.9
	2	1987.5	1140.7	2003.2	2.304	2.493	7.6
	3	1985.4	1134.2	2002.1	2.288	2.493	8.2
H-60G-GP	1	1984.9	1133.9	1999.9	2.292	2.493	8.1
	2	1981.7	1133	1992	2.307	2.493	7.5
	3	1984.2	1137.9	2001.6	2.297	2.493	7.9

- Intermediate Temperature Cracking Resistance (continued)

Mix ID	Specimen	W_{Dry} (g)	W_{soak} (g)	W_{SSD} (g)	G_{mb} (-)	G_{mm} (-)	AV (%)
H-60L-L	1	1888.8	1052.1	1900.2	2.227	2.343	5.0
	2	1891.6	1049	1902	2.218	2.343	5.3
	3	1890.8	1050.6	1902.4	2.220	2.343	5.2
H-60L-LO	1	1889.9	1048	1898.3	2.223	2.343	5.1
	2	1890.2	1057.8	1905.4	2.230	2.343	4.8
	3	1891.9	1054.2	1897.3	2.244	2.343	4.2
H-60L-LP	1	1887.9	1052.6	1898.8	2.231	2.343	4.8
	2	1888.4	1050.5	1897.6	2.229	2.343	4.9
	3	1888.7	1054.6	1897.4	2.241	2.343	4.4
H-60L-GO	1	1942.1	1117.4	1957.7	2.311	2.460	6.1
	2	1939.9	1109.2	1955.5	2.292	2.460	6.8
	3	1941.3	1107.7	1954.7	2.292	2.460	6.8
H-60L-GP	1	1942.7	1101.8	1957.9	2.269	2.460	7.8
	2	1940.1	1098.3	1954.4	2.266	2.460	7.9
	3	1939.2	1109.2	1961.8	2.274	2.460	7.6

Determination of Vacuum Saturation

- Moisture Susceptibility Specimens

Mix ID	Specimen	V_{VA} (cm^3)	Vacuum W_{SSD} (g)	V_{WA} (cm^3)	P_{st} (%)
H-60G-G	1	48.1	1565	23.6	0.49
	2				
	3				
	4	44.6	1424.2	25.9	0.58
	5	47.1	1485.9	22.7	0.48
	6				
	7				
	8				
H-60G-GO	1				
	2				
	3	50.0	1481.3	27.3	0.55
	4	51.0	1536	28.7	0.56
	5				
	6				
	7				
	8	49.8	1458.8	26	0.52

- Moisture Susceptibility Specimens (continued)

Mix ID	Specimen	V_{VA} (cm^3)	Vacuum W_{SSD} (g)	V_{WA} (cm^3)	P_{st} (%)
H-60G-GP	1	48.2	1528.8	27.2	0.56
	2	46.6	1482.4	24.5	0.53
	3				
	4				
	5				
	6				
	7				
	8	47.1	1457.3	28.9	0.61
H-60L-L	1	28.4	1420.4	12	0.42
	2				
	3	26.8	1435.6	12.5	0.47
	4	27.3	1406.8	14.7	0.54
	5				
	6				
	7				
	8	27.8	1442.1	15.1	0.54
H-60L-LO	1				
	2	27.3	1373.7	16.8	0.62
	3				
	4	27.0	1370.8	15.6	0.58
	5				
	6				
	7				
	8	26.1	1377.8	14.4	0.55
H-60L-LP	1				
	2				
	3	28.4	1391.6	13.2	0.47
	4	25.8	1456.7	13.4	0.52
	5				
	6				
	7				
	8	26.1	1375.4	11	0.42
H-60L-GO	1	46.8	1426.5	24.8	0.53
	2				
	3				
	4				
	5				
	6	43.9	1404.5	22	0.50
	7				
	8	46.7	1495.8	25.6	0.55
H-60L-GP	1				
	2				
	3				
	4	47.4	1523.2	22.9	0.48
	5				
	6				
	7	44.4	1427.4	28.2	0.63
	8	48.6	1517.4	29.7	0.61

B. Cold Recycling with Emulsion

Determination of Maximum Specific Gravity (G_{mm})

Mix ID	$W_{mix\text{-}loose}$ (g)	$W_{pyc\ (soak)}$ (g)	$W_{spyc+mix\ (soak)}$ (g)	G_{mm} (-)
C-80L-LE	1420.7	1513.2	2326.6	2.339
C-100L-E	1251.3	1513.2	2220.5	2.300
C-60G-GE	1386.6	1513.2	2349.8	2.521
C-60L-GE	1396.5	1513.2	2344.3	2.470
C-60L-LE	1392.7	1489.3	2293.2	2.365
C-80G-GE	1388.0	1489.3	2312.1	2.456

Determination Bulk Specific Gravity (G_{sb})

- Moisture Susceptibility Specimens

Mix ID	Specimen	Height (mm)					W_{Dry} (g)	W_{soak} (g)	W_{SSD} (g)	G_{mb} (-)	G_{mm} (-)	AV (%)
		1	2	3	4	Average						
C-60L-LE	1	40.53	40.49	39.97	40.21	40.30	1411.4	727.1	1419.7	2.038	2.365	13.8
	2	40.44	40.58	40.53	40.09	40.41	1411.7	724.8	1419.9	2.031	2.365	14.1
	3	40.52	40.29	40.19	40.53	40.38	1411.4	729.9	1422.4	2.038	2.365	13.8
	4	40.44	40.29	39.99	40.01	40.18	1414.4	733.6	1424.7	2.047	2.365	13.5
	5	40.75	40.49	40.27	40.25	40.44	1413.6	730.6	1422.5	2.043	2.365	13.6
	6	41.06	41.05	40.92	40.86	40.97	1411.7	737.8	1427.2	2.048	2.365	13.4
	7	40.08	40.32	40.45	40.09	40.24	1415.6	736.3	1428.6	2.045	2.365	13.5
	8	40.06	40.05	40.19	40.18	40.12	1395.3	716.3	1407	2.020	2.365	14.6
C-80L-LE	1	40.13	39.99	39.72	39.49	39.83	1401.5	726.3	1408.9	2.053	2.339	12.2
	2	40.47	40.2	40.36	40.4	40.36	1402.5	733.6	1416.6	2.053	2.339	12.2
	3	41.11	40.75	40.78	40.89	40.88	1435.5	746.5	1443.2	2.060	2.339	11.9
	4	41.02	40.63	40.63	40.78	40.77	1407.1	733.9	1420.8	2.048	2.339	12.5
	5	40.4	40.55	40.44	40.07	40.37	1421.7	742.5	1430.3	2.067	2.339	11.6
	6	40.15	40.05	39.75	39.84	39.95	1399.6	723.5	1407.1	2.047	2.339	12.5
	7	40.2	40.12	40.29	40.35	40.24	1402.7	729	1413.5	2.049	2.339	12.4
	8	40.44	40.89	40.78	40.42	40.63	1405.8	737.3	1423.3	2.049	2.339	12.4

- Moisture Susceptibility Specimens (continued)

Mix ID	**Specimen**	**Height (mm)**					**W_{Dry} (g)**	**W_{soak} (g)**	**W_{SSD} (g)**	**G_{mb} (-)**	**G_{mm} (-)**	**AV (%)**
		1	**2**	**3**	**4**	**Average**						
C-100L-E	1	38.9	38.96	38.93	38.63	38.86	1302.2	647	1314.7	1.950	2.300	15.2
	2	39.08	38.71	38.67	38.72	38.80	1301.5	644.6	1312.1	1.950	2.300	15.2
	3	38.89	38.87	38.85	38.91	38.88	1302.3	639.9	1310.1	1.943	2.300	15.5
	4	38.98	38.8	38.19	38.6	38.64	1301.4	647.7	1313.2	1.956	2.300	15.0
	5	38.62	38.45	38.89	38.8	38.69	1302.2	646.5	1313.5	1.952	2.300	15.1
	6	38.67	38.91	38.67	38.5	38.69	1301.1	647	1313.2	1.953	2.300	15.1
	7	39.12	38.95	38.53	38.79	38.85	1304.5	651.5	1317.8	1.958	2.300	14.9
	8	37.59	37.38	37.57	37.74	37.57	1253.9	613.1	1262.8	1.930	2.300	16.1
C-60G-GE	1	40.41	40.42	40.01	40.4	40.31	1382.1	762.2	1435.8	2.052	2.521	18.6
	2	40.3	40.03	40.6	40.13	40.27	1384.1	765.4	1435.7	2.065	2.521	18.1
	3	40.5	40.51	41.28	40.83	40.78	1387.7	762.3	1437.5	2.055	2.521	18.5
	4	40.55	40.24	40.48	40.22	40.37	1388.4	772.1	1443.1	2.069	2.521	17.9
	5	40.53	40.42	40.14	40.17	40.32	1387.1	763	1439.8	2.049	2.521	18.7
	6	40.25	40.13	39.97	40.23	40.15	1384.9	761.8	1438.2	2.047	2.521	18.8
	7	40.14	39.95	40.05	40.11	40.06	1386.7	760.2	1439.3	2.042	2.521	19.0
	8	39.85	39.51	39.88	39.72	39.74	1384.4	757.8	1434.9	2.045	2.521	18.9
C-80G-GE	1	41.38	41.45	41.35	41.42	41.40	1386.8	739.7	1440.2	1.980	2.456	19.4
	2	41.72	41.33	41.38	41.24	41.42	1388.6	750.4	1448.4	1.989	2.456	19.0
	3	41.31	41.36	41.21	41.3	41.30	1389.6	757.3	1453.1	1.997	2.456	18.7
	4	41.38	41.42	41.37	41.39	41.39	1389.1	756.3	1446.3	2.013	2.456	18.0
	5	41.16	41.49	41.23	41.47	41.34	1388.3	749.3	1444.3	1.998	2.456	18.6
	6	41.18	41.6	41.72	41.37	41.47	1388.5	749	1447.9	1.987	2.456	19.1
	7	41.15	41.18	41.36	41.31	41.25	1384.8	746.7	1446.2	1.980	2.456	19.4
	8	41.46	41.32	41.4	41.47	41.41	1389.1	751.6	1444	2.006	2.456	18.3
C-60L-GE	1	39.21	39.53	39.02	39.22	39.25	1398.4	737.8	1409.2	2.083	2.470	15.7
	2	39.87	39.68	39.43	39.65	39.66	1398.8	745.7	1417.2	2.083	2.470	15.7
	3	39.83	39.54	39.22	39.15	39.44	1393.6	741.3	1409.6	2.085	2.470	15.6
	4	39.48	39.32	39.04	39.37	39.30	1397.2	738.1	1409.9	2.080	2.470	15.8
	5	39.44	39.27	39.39	39.3	39.35	1399.5	739.7	1414	2.075	2.470	16.0
	6	39.53	39.52	39.25	39.3	39.40	1398.2	736.9	1412.8	2.069	2.470	16.2
	7	39.58	39.41	39.37	39.33	39.42	1398.2	739.6	1411.9	2.080	2.470	15.8
	8	39.46	39.54	39.85	39.7	39.64	1400.6	740.9	1415.9	2.075	2.470	16.0

- Rutting Resistance Specimens

Mix ID	Specimen	W_{Dry} (g)	W_{soak} (g)	W_{SSD} (g)	G_{mb} (-)	G_{mm} (-)	AV (%)
C-60L-LE	1	2390.4	1248.4	2399.4	2.077	2.365	12.2
	2	2356.6	1240.3	2370.1	2.086	2.365	11.8
	3	2352.6	1224.9	2364.5	2.064	2.365	12.7
	4	2348.6	1221.8	2360.9	2.062	2.365	12.8
C-80L-LE	1	2223.9	1230.5	2333.4	2.016	2.339	13.8
	2	2294.9	1194.2	2300.7	2.074	2.339	11.3
	3	2346.0	1247.9	2355.3	2.118	2.339	9.5
	4	2344.0	1250.3	2351.6	2.128	2.339	9.0
C-100L-E	1	2175.8	1089.1	2189.9	1.977	2.300	14.1
	2	2177.0	1091.6	2192.9	1.977	2.300	14.1
	3	2176.0	1096.6	2195.7	1.980	2.300	13.9
	4	2154.5	1075.5	2173.2	1.963	2.300	14.7
C-60G-GE	1	2327.5	1272.6	2366.4	2.128	2.521	15.6
	2	2328.0	1282.1	2390.5	2.100	2.521	16.7
	3	2325.0	1282.7	2391.6	2.097	2.521	16.8
	4	2329.1	1293.6	2380.3	2.143	2.521	15.0
C-80G-GE	1	2323.7	1266.1	2389.2	2.069	2.456	15.7
	2	2343.8	1271.4	2391.7	2.092	2.456	14.8
	3	2321.8	1274.6	2373.0	2.114	2.456	13.9
	4	2320.6	1265.4	2391.0	2.062	2.456	16.0
C-60L-GE	1	2330.6	1236.9	2356.2	2.082	2.470	15.7
	2	2267.4	1172.2	2299.3	2.012	2.470	18.5
	3	2329.1	1248.4	2344.3	2.125	2.470	14.0
	4	2330.8	1252.2	2346.6	2.130	2.470	13.8

- Raveling Resistance Specimens

Mix ID	Specimen	W_{Dry} (g)	W_{soak} (g)	W_{SSD} (g)	G_{mb} (-)	G_{mm} (-)	AV (%)
C-60L-LE	1	4245.3	2241.7	4260.5	2.103	2.365	11.1
	2	4244.3	2242.3	4257.9	2.106	2.365	11.0
	3	4219.5	2195.1	4234.9	2.069	2.365	12.5
C-80L-LE	1	4159.6	2174.9	4211.5	2.042	2.339	12.7
	2	4192	2001.1	4219.1	1.890	2.339	19.2
	3	4159.6	2166.9	4185.3	2.061	2.339	11.9
C-100L-E	1	3920	1976.6	3994.8	1.942	2.300	15.6
	2	3930.4	1998.6	3998.1	1.966	2.300	14.5
	3	3901.6	1975.4	3963.1	1.963	2.300	14.7

- Raveling Resistance Specimens (continued)

Mix ID	Specimen	W_{Dry} (g)	W_{soak} (g)	W_{SSD} (g)	G_{mb} (-)	G_{mm} (-)	AV (%)
C-60G-GE	1	4189.4	2355.1	4338.6	2.112	2.521	16.2
	2	4184.1	2338.6	4326.2	2.105	2.521	16.5
	3	4166	2351.6	4327.2	2.109	2.521	16.3
C-80G-GE	1	4165.4	2312.2	4306	2.089	2.456	14.9
	2	4175.5	2294.8	4286.7	2.096	2.456	14.6
	3	4173.8	2314.1	4307.3	2.094	2.456	14.7
C-60L-GE	1	4216.5	2288.2	4277.4	2.120	2.470	14.2
	2	4212.4	2275.2	4265.2	2.117	2.470	14.3
	3	4228	2299	4290.4	2.123	2.470	14.0

Determination of Vacuum Saturation

- Moisture Susceptibility Specimens

Mix ID	Specimen	V_{VA} (cm^3)	Vacuum W_{SSD} (g)	V_{WA} (cm^3)	P_{st} (%)
C-60L-LE	1				
	2	100.9	1478.1	66.4	0.66
	3				
	4	95.6	1477.9	63.5	0.66
	5				
	6	97.1	1475.5	63.8	0.66
	7	96.3	1480.5	64.9	0.67
	8				
C-80L-LE	1	86.2	1455.3	53.8	0.62
	2	87.3	1460.2	57.7	0.66
	3	86.3	1490.1	54.6	0.63
	4	89.7	1465.8	58.7	0.65
	5				
	6				
	7				
	8				
C-100L-E	1				
	2				
	3				
	4	102.2	1372.6	71.2	0.70
	5				
	6	103.2	1371.8	70.7	0.69
	7	102.1	1372.5	68	0.67
	8				

- Moisture Susceptibility Specimens (continued)

Mix ID	Specimen	V_{VA} (cm^3)	Vacuum W_{SSD} (g)	V_{WA} (cm^3)	P_{st} (%)
C-60G-GE	1	132.5	1460.3	78.2	0.59
	2	128.7	1473.9	89.8	0.70
	3	133.2	1474.1	86.4	0.65
	4				
	5	133.4	1473.9	86.8	0.65
	6				
	7				
	8				
C-80G-GE	1				
	2	139.1	1489.5	100.9	0.73
	3				
	4				
	5				
	6				
	7				
	8	134.0	1473.6	84.5	0.63
C-60L-GE	1	108.6	1465.3	66.9	0.62
	2	109.8	1465.7	66.9	0.61
	3	108.6	1458.9	65.3	0.60
	4				
	5				
	6				
	7				
	8	112.0	1484.6	84	0.75

C. Cold Recycling with Foamed Asphalt

Determination of Maximum Specific Gravity (Gmm)

Mix ID	$W_{mix\text{-}loose}$ (g)	$W_{pyc\ (soak)}$ (g)	$W_{spyc+mix\ (soak)}$ (g)	G_{mm} (-)
C-60L-LF	2318.6	1488.7	2828.2	2.368
C-80L-LF	2626.5	1512.6	3013.8	2.334
C-100L-F	2581.9	1488.7	2912.9	2.230
C-60L-GF	2659.1	1512.6	3071.2	2.416

Determination Bulk Specific Gravity (G_{sb})

- Moisture Susceptibility Specimens

Mix ID	Specimen	Height (mm)					W_{Dry} (g)	W_{soak} (g)	W_{SSD} (g)	G_{mb} (-)	G_{mm} (-)	AV (%)
		1	2	3	4	Average						
C-60L-LF	1	39.74	39.25	40.58	42.15	40.43	1391.7	738.6	1429.6	2.014	2.368	15.0
	2	39.42	41.22	41.34	39.6	40.40	1378.0	726.9	1415.0	2.003	2.368	15.4
	3	38.79	42.23	41.78	39.02	40.46	1392.1	733.7	1426.6	2.009	2.368	15.2
	4	39.34	40.83	40.77	40.03	40.24	1371.7	722.0	1405.1	2.008	2.368	15.2
	5	40.32	39.29	37.22	38.37	38.80	1325.5	703.5	1361.6	2.014	2.368	15.0
	6	37.28	38.37	40.05	38.94	38.66	1326.4	699.6	1358.5	2.013	2.368	15.0
	7	36.03	38.12	40.91	39.09	38.54	1318.5	694.9	1351.8	2.007	2.368	15.2
	8	40.07	39.4	39.17	38.86	39.38	1344.1	707.5	1376.5	2.009	2.368	15.2
C-80L-LF	1	40.81	39.96	39.32	39.61	39.93	1345.4	696.8	1376.1	1.981	2.334	15.1
	2	39	39.62	39.08	38.9	39.15	1320.6	689.4	1352.5	1.992	2.334	14.7
	3	42.24	40.43	40.6	41.63	41.23	1397.4	734.1	1437.5	1.987	2.334	14.9
	4	39.82	39.02	37.89	38.28	38.75	1312.8	688.9	1345.6	1.999	2.334	14.4
	5	40.46	39.82	38.91	39.97	39.79	1347.5	697.9	1378.3	1.980	2.334	15.2
	6	40.6	40.65	40.76	40.58	40.65	1349.4	704.4	1382.0	1.991	2.334	14.7
	7	39.1	37.07	37.76	40.11	38.51	1286.6	675.9	1325.3	1.981	2.334	15.1
	8	41,98	41.05	39.73	41.36	40.71	1390.9	729.8	1430.4	1.985	2.334	15.0
C-100L-F	1	36.52	37.73	36.71	36.35	36.83	1217.5	619.3	1242.3	1.954	2.230	12.4
	2	38.38	39.98	40.75	38.33	39.36	1308.7	664.5	1335.7	1.950	2.230	12.6
	3	37.18	35.63	34.39	35.88	35.77	1180.2	594.4	1202.0	1.942	2.230	12.9
	4	36.41	36.45	34.94	34.82	35.66	1176.8	595.6	1199.6	1.948	2.230	12.7
	5	37.74	37.44	38.07	38.55	37.95	1273.3	647.7	1299.8	1.953	2.230	12.4
	6	35.44	36.32	35.33	34.18	35.32	1185.5	602.0	1211.2	1.946	2.230	12.7
	7	40.01	40.05	38.45	38.03	39.14	1314.1	661.8	1335.0	1.952	2.230	12.5
	8	37.88	38.22	39.7	39.31	38.78	1305.6	660.7	1330.9	1.948	2.230	12.7
C-60L-GF	1	39.04	39.3	39.31	39.24	39.22	1375.2	746.5	1417.9	2.048	2.416	15.2
	2	36.77	37.04	37.7	37.76	37.32	1319.2	709.2	1352.3	2.051	2.416	15.1
	3	36.4	37	37.46	36.98	36.96	1311.4	703.3	1341.5	2.055	2.416	15.0
	4	33.43	34.97	36.35	35.17	34.98	1241.3	673.5	1278.5	2.052	2.416	15.1
	5	37.81	37.81	37.81	37.94	37.84	1296.0	703.5	1336.3	2.048	2.416	15.2
	6	39.75	38.96	38.49	39.21	39.10	1382.8	743.4	1416.1	2.056	2.416	14.9
	7	39.91	39.3	39.41	40.04	39.67	1393.5	747.4	1425.5	2.055	2.416	15.0
	8	41.79	39.57	39.69	41.52	40.64	1435.8	779.0	1474.2	2.065	2.416	14.5

- Rutting Resistance Specimens

Mix ID	**Specimen**	**W_{Dry} (g)**	**W_{soak} (g)**	**W_{SSD} (g)**	**G_{mb} (-)**	**G_{mm} (-)**	**AV (%)**
C-60L-LF	1	2378.9	1295.9	2402.4	2.150	2.368	9.2
	2	2390.4	1292.9	2410.2	2.139	2.368	9.7
	3	2391.7	1284.8	2406.6	2.132	2.368	10.0
	4	2381.2	1295.1	2400.0	2.155	2.368	9.0
	5	2394.7	1295.0	2413.6	2.141	2.368	9.6
C-80L-LF	1	2389.7	1284.7	2402.2	2.138	2.334	8.4
	2	2377.0	1271.8	2388.1	2.129	2.334	8.8
	3	2382.3	1287.5	2393.7	2.154	2.334	7.7
	4	2377.5	1277.7	2386.7	2.144	2.334	8.1
C-100L-F	1	2215.8	1140.5	2254.9	1.988	2.230	10.9
	2	2209.9	1141.5	2254.5	1.986	2.230	10.9
	3	2210.5	1137.0	2245.4	1.994	2.230	10.6
	4	2204.7	1124.9	2236.8	1.983	2.230	11.1
C-60L-GF	1	2391.3	1307.0	2421.3	2.146	2.416	11.2
	2	2383.3	1302.1	2414.4	2.143	2.416	11.3
	3	2387.0	1308.7	2416.6	2.155	2.416	10.8
	4	2375.5	1295.9	2413.6	2.125	2.416	12.1

- Raveling Resistance Specimens

Mix ID	**Specimen**	**W_{Dry} (g)**	**W_{soak} (g)**	**W_{SSD} (g)**	**G_{mb} (-)**	**G_{mm} (-)**	**AV (%)**
C-60L-LF	1	4303.9	2350.7	4361.2	2.141	2.368	9.6
	2	4305.5	2323.5	4339.7	2.135	2.368	9.8
	3	4318.6	2345.2	4358.6	2.145	2.368	9.4
C-80L-LF	1	4308.4	2313.7	4333.2	2.133	2.334	8.6
	2	4306.7	2317.5	4330.9	2.139	2.334	8.4
	3	4299.9	2307.4	4325.1	2.131	2.334	8.7
C-100L-F	1	3977.5	2058.5	4063.6	1.984	2.230	11.0
	2	3986	2037.3	4039.9	1.990	2.230	10.8
	3	3984.9	2058.2	4063.6	1.987	2.230	10.9
C-60L-GF	1	4276	2381.2	4384.5	2.134	2.416	11.7
	2	4283.4	2371.6	4365.5	2.148	2.416	11.1
	3	4299.5	2367.5	4375.8	2.141	2.416	11.4

Determination of Vacuum Saturation

- Moisture Susceptibility Specimens

Mix ID	Specimen	V_{VA} (cm^3)	Vacuum W_{SSD} (g)	V_{WA} (cm^3)	P_{st} (%)
C-60L-LF	1	106.8	1466.2	74.5	0.70
	2				
	3	108.4	1465.9	73.8	0.68
	4				
	5	102.5	1401.2	75.7	0.74
	6	102.4	1398.7	72.3	0.71
	7				
	8				
C-80L-LF	1				
	2	101.4	1390.6	70	0.69
	3	108.3	1477.2	79.8	0.74
	4	98.3	1381.3	68.5	0.70
	5				
	6	105.6	1446.2	96.8	0.92
	7				
	8				
C-100L-F	1	80.6	1285.9	68.4	0.85
	2	87.4	1383.4	74.7	0.85
	3				
	4				
	5	83.4	1336.7	63.4	0.76
	6				
	7	86.3	1371.1	57	0.66
	8				
C-60L-GF	1				
	2				
	3	97.7	1386.3	74.9	0.77
	4				
	5				
	6	103.0	1443.8	61	0.59
	7	104.8	1480.2	86.7	0.83
	8	104.4	1505.5	69.7	0.67