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EVALUATE RECYCLED CONCRETE AS HOT MIX ASPHALT AGGREGATE

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Each year around 200 million tons of demolition waste is produced from aging US infrastructures out of which 100 million tons are Portland cement concrete debris. The lack of landfill areas, environmental regulations and costs have hindered safe disposal of this waste. This led to seeking alternate ways of recycling this demolition waste. Recycling the concrete waste not only reduces the waste disposal problem, but also reduces the amount of quarrying of virgin aggregate. This study evaluated the effects of recycled concrete aggregates on mix design and performance of HMA. It was found that the use of recycled concrete aggregates to replace virgin aggregates increased the asphalt content needed in the mix, due to the high absorption of recycled concrete aggregate. In addition, with the increase of content of recycled concrete aggregate, the resistance to fatigue, rutting, thermal cracking and moisture damage is reduced. Therefore, cautions should be made to use recycled concrete aggregate in HMA, even though the volumetric requirements are met. The Superpave volumetrics-based mix design is not sufficient to capture the performance of mixes and should be supplemented with performance-based tests. The concept of “effective” asphalt content during a mix design might not be accurate. The absorbed asphalt may play a role in the performance of HMA. Further studies are needed to test more sources of recycled concrete aggregates in HMA and to verify the shortcomings of volumetrics-based mix design.</td></tr>
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CHAPTER 1. INTRODUCTION

Portland cement concrete has been used as one of the major sources of construction materials for buildings, roads, bridges, runways and other structures. It primarily consists of cement, aggregate, and water. Aggregate is a major structural component of concrete and is typically obtained from natural stone or quarry. In 2008, 1.04 billion metric tons of sand and gravel were produced for construction in the United States [1]. Increased environmental awareness has limited the quarrying of virgin aggregate areas. Also each year around 200 million tons of demolition waste is produced from aging US infrastructure out of which 100 million tons are concrete debris [2]. Disposal of this waste in landfills has been a traditional solution. The lack of landfill areas, environmental regulations and costs have hindered safe disposal of these wastes. This led to seeking alternate ways of reusing this demolition waste by recycling it. Recycling the concrete waste not only reduces the waste disposal problem, but also reduces the amount of quarrying of virgin aggregate. Recycled concrete aggregate (RCA) is produced by properly crushing and sieving the demolished waste to obtain required size of aggregates that will substitute the need for virgin aggregates. So far the use of recycled concrete has been limited in the construction of concrete structures (rigid pavement, building and runways) or use as pavement base.

Recycled concrete aggregates are different from virgin aggregates due to the amount of cement paste remaining on the surface of the recycled aggregates after undergoing the recycling process [3, 4, and 5]. The presence of cement paste increases the porosity of the aggregates, reduces the particle density, and thus, the quality and water absorption capacity of RCA vary. It has been reported that RCA in HMA affected the volumetric properties and performance of HMA [3, 4, 5, and 6].

1.1 LITERATURE REVIEW

Illinois department of transportation (IDOT) conducted a field research study on recycling an existing badly "D" cracked 6.89-inch thick continuously reinforced concrete pavement into a 16-inch thick full depth asphalt concrete inlay [3]. The section constructed with HMA containing RCA had higher backcalculated elastic moduli than the one constructed with virgin aggregate only. The moisture susceptibility test results, based on tensile strength ratio, showed that HMA with RCA is more resistant to stripping than virgin aggregates due to higher alkalinity.

Wong et al. studied the use of the RCA as a partial aggregate substitution in HMA [4]. Three HMA mixes were included in the study by substituting granite filler/fines with 6% untreated, 45% untreated, and 45 % heat-treated recycled concrete, respectively. All three mixes passed the wearing course criteria specified by the Singapore Land Transport Authority, based on the Marshall mix design method. The performance tests on mix with 6% RCA showed comparable resilient modulus and creep resistance to those of the traditional HMA mix. The mixes with the higher percentage of RCA showed higher resilient modulus and resistance to creep.

Paranavithana et al. performed experiments on the effects of recycled concrete aggregates on properties of HMA [5]. 50% RCA by the dry weight of total aggregate was used as coarse aggregate in HMA. The performance tests carried out on these two mixes showed that the use of RCA in HMA lowered the resilient modulus and creep resistance of the mix. The use of RCA in HMA increased the stripping potential of mix. Also the mixes containing RCA showed large variations of strength under dry and wet conditions.

Topal et al. found that RCA can substitute of HMA aggregate to achieve the required Marshall stability and indirect tensile strength of the mixtures [6]. Different percentages of RCA were blended with limestone aggregates. These mixes showed differences in volumetric properties. The Marshall stability values increased with the increase of RCA in the mix. However, the voids in mineral aggregate (VMA) and the voids filled with asphalt (VFA) decreased with the increase of RCA content. This was believed due to crushing of RCA by the Marshall compactor during compaction [6]. The specific gravity of mixes decreased with the increase in the amount of RCA. The tensile strength of mix containing RCA was found to be higher than that of the control mix which was believed because the internal friction of RCA is higher than that of natural limestone aggregates [6]. However, RCA was not recommended for use in the wearing course due to RCA's susceptibility to abrasion by the vehicles.

Beale et al. (2008) performed a study to investigate the feasibility of using RCA for a low-volume traffic road in Michigan [7]. 25%, 35%, 50%, and 75% of virgin aggregates by the weight of total aggregates were substituted with RCA. It was found that increasing the percentage of RCA decreased the VMA and VFA of the mixes. The laboratory test results indicated that all 4 mixes with RCA passed the minimum rutting specification of 0.32 inch rut depth. Dynamic modulus test results showed that stiffness of mixes with RCA were less than that of the control mix. For the moisture susceptibility tests, all the mixes, except for the 75% RCA mix, passed the tensile strength ratio of 80%. The compaction energy test showed that using the RCA in HMA reduced the energy needed for compaction.

Table 1 below shows the summarized results of RCA performance tests done by others [3, 4, 5, 6, and 7]. It can be seen that the findings were inconsistent and largely depended on the sources of RCA used. In addition, the laboratory tests employed by the previous researchers may not be true performance tests which can directly predict the performance of mixes. Therefore, Study is needed to address RCA physical properties (specific gravity, absorption) and their effects on mix design properties (volumetrics) and performance of mix, based on recent development of performance tests.

Table 1. Literature Review Summary of RCA Test Results

Researcher	RCA (%)	Stripping Resistance	Resilient Modulus	Dynamic Modulus	Back-Calculated Elastic Modulus	Creep Resistance	Indirect Tensile Strength	Marshal Stability	Rutting
Schutzbach [3]	50	Increased	N/A	N/A	Increased	N/A	N/A	N/A	NA
Wong et al. [4]	6 and 45	N/A	Increased	N/A	N/A	Increased	N/A	N/A	NA
Paranavithana et al. [5]	50	Reduced	Reduced	N/A	N/A	Reduced	N/A	N/A	NA
Topal et al. [6]	10, 20 and 30	N/A	N/A	N/A	N/A	N/A	Increased	Increased	N/A
Beale et al. [7]	25, 35, 50 , and 75	Reduced	N/A	Reduced	N/A	N/A	N/A	N/A	Increased

1.2 RESEARCH OBJECTIVES

The objectives of this research were to determine the feasibility of using RCA as HMA aggregate. This study was carried out to evaluate the following effects of RCA on:

(i) Superpave mix design,

(ii) Performance behaviors of HMA (rutting, fatigue cracking, thermal cracking, and moisture susceptibility).

CHAPTER 2. MIX DESIGN OF HMA CONTAINING RCA

2.1 INTRODUCTION

Most of highway agencies and/or contractors design hot mix asphalt in accordance with the Superpave mix design method, especially the volumetric design [8]. Superpave mix design method was originally developed based on the virgin aggregates. The effects of the use of RCA on volumetric design of HMA need to be studied. This Chapter presented the results of Superpave mix design on HMA containing difference percentages of RCA.

2.2 MATERIALS

Recycled concrete aggregates from Renton Concrete Recycling (Renton, WA) and Contractors Concrete Recycling (Seattle, WA) were used as two sources of RCA in the study. The RCA from Renton Concrete Recycling was referred to as RCA source 1 and the one from Contractors Concrete Recycling was referred to as RCA source 2. To produce RCA, concrete infrastructures are first demolished and broken into large chunks. These large chunks of concrete debris are then transported into nearby concrete recycling site. The steel bar remained in the debris is subsequently removed and concrete debris is further crushed into smaller aggregate size required for constructions. The virgin aggregates (basalt) used in the study were provided by the POE Asphalt Inc., located in Pullman, WA and the properties meet the WSDOT specifications on HMA aggregates. The asphalt binder used in the study was PG 58-28 provided by Idaho Asphalt Supply, Inc. Figures 1 and 2 show the coarse aggregates of RCA sources 1 and 2, respectively.

Figure 1. Renton Recycled Concrete (Source 1)

Figure 2. Contractor Recycled Concrete Aggregate (Source 2)

2.3 PROPERTIES OF AGGREGATE

The consensus and source properties of RCA and virgin aggregate were evaluated, respectively, in accordance with the Washington Department of Transportation (WSDOT) specifications and test methods. Table 2 shows the properties of RCA and virgin aggregates used in the study. It was found that 100% RCA (both sources 1 and 2) met most of the specifications, but failed the

WSDOT degradation factor requirements. The WSDOT degradation test determines the susceptibility of an aggregate to degradation into clay-like plastic fines in the presence of moisture. The aggregates in the presence of moisture are degraded by oscillating the steel canister at a rate of 300 (±5) revolutions per minute. This test is similar to the Micro-Deval test, except that there are no steel balls used in the degradation test. Figure 3 shows the degradation test device. To determine the percentage of RCA at which the blend of RCA and virgin aggregate passes the WSDOT degradation value requirements, attempt was made by preparing a blend containing 80% RCA and 20% virgin aggregate. It was found that the blend of 80% RCA and 20% virgin aggregates met the WSDOT specifications for degradation value for both sources of RCA. Therefore, other blends (20%, 40%, and 60%) was not tested. Table 3 shows the results of degradation tests using 80% RCA mixed with 20% virgin aggregates.

Figure 3. Aggregate Degradation Machine

Table 2. Properties of Virgin and RCA Aggregate

Test	Test Standards	Properties			WSDOT Specifications
		Virgin Aggregate	RCA 1	RCA 2	
L. A Abrasion	AASHTO 96	20	22	24	Max. 30
Degradation Factor	WSDOT 113	61	15	13	Min. 30
Fractured Faces	WSDOT FOP/AASTHO TP 61	95%	96%	93%	Min. 90%
Flat/ Elongated	WSDOT FOP/ASTM D4791	3%	1%	0.5%	Max. 10%
Sand Equivalent	WSDOT FOP For AASHTO T 176	90	75	80	Min. 45%
Uncompacted Void Content	AASTHO T 304 and ASTM C 1252	45%	42%	41%	Min. 40%
Bulk Specific Gravity Coarse Aggregate	AASTHO T-85	2.675	2.412	2.427	N/A
Bulk Specific Gravity Fine Aggregate	AASTHO T-84	2.686	2.092	2.125	N/A

Table 3. Degradation Test Results of 80% RCA Blended with 20% Virgin Aggregate

RCA Source	Degradation Value	WSDOT Specifications
RCA 1	37	Min 30
RCA 2	33	Min 30

2.4 HMA MIX DESIGN

Laboratory mixing and compaction of HMA were performed in accordance with AASTHO R 28 and T323 [9]. Five different percentages of RCA (20%, 40%, 60%, 80%, and 100%) were blended with virgin aggregate. Also the control HMA mix design was performed using 100% virgin aggregate and was named as “RCA 0%” mix. For all the mix designs, the nominal maximum size of aggregate was ½". The design equivalent single axle load (ESALs) were 3-10 millions. The N_{ini}, N_{des}, and N_{max} were 8, 100, and 158, respectively, in accordance with the WSDOT specifications [8]. The samples were mixed using drum mixer. During mixing it was found that mixes containing RCA were less workable compared to the mixes containing virgin aggregates only. The mixing temperature for RCA mixes was 307°F. The mixes were aged for two hours prior to compaction. An Interlaken gyratory compactor was used for compaction and the compaction temperature was 284°F. The gradation of each mix was modified for each mix to meet the WSDOT volumetric specifications. Table 4 shows the gradations of RCA 1 and 2, and the virgin aggregates.

Table 4. Source Gradations of RCA 1, 2 and Virgin Aggregates

Sieve Size	% Passing				
	RCA 1	RCA 2	Virgin Aggregates		
			5/8"	3/8"	1/4"
3/4"	100	95.0	100	100	100
1/2"	91.2	82.1	94	100	100
3/8"	77.4	69.3	55	96	100
# 4	50.3	51.4	4	14	85
# 8	31.7	37.8	1	3	49
# 16	21.4	30.1	1	2	28
# 30	14.6	23.3	1	2	18
# 50	8.4	16.4	1	2	14
# 100	6.3	11.8	1	2	11
# 200	4.4	9.9	0.7	1.6	9

2.5 HMA MIX DESIGN RESULTS AND ANALYSIS

Tables 5 and 6 show the gradations and volumetrics of mix designs of HMA containing RCA 1, and Tables 7 and 8 for HMA containing RCA 2. The mix design results showed that the addition of RCA increased the asphalt content needed. Figure 4 show the relationship between the optimum asphalt content (AC) percent and RCA percentage. It indicates that the asphalt content increased linearly with the increase of RCA %. This is due to the fact that RCA aggregates are highly absorptive, especially for RCA 1. For RCA 1, the absorption rate of coarse aggregates is 4.5% and 4.6% for fine aggregates. For RCA 2, the absorption rate of coarse aggregates is 4.0% and 9.0% for fine aggregates. The bulk specific gravity (G_{sb}) of both RCAs was lower than that of virgin aggregates, especially for the fine aggregate. The bulk specific gravity of RCA 1 was 2.092, 2.125 for RCA 2, and the 2.686 for virgin aggregates. Also, the mix design results showed that the addition of RCA reduced the bulk specific gravity (G_{mb}) and theoretical maximum specific gravity (G_{mm})of the mix, as illustrated in Figures 5 and 6.

Table 5. Gradations of Blends of RCA 1 and Virgin Aggregates

Sieve size	Percent Passing for Different RCA Percentages						Gradation Control Point
	0%	20%	40%	60%	80%	100%	
3/4"	100.0	100	100	100	100	100.0	
1/2"	98.0	97.8	97.8	97.8	97.8	91.2	90-100
3/8"	88.0	86.7	86.7	86.7	86.7	77.2	90 max
# 4	61.0	57.4	57.4	57.4	57.4	50.3	
# 8	45.0	33.4	33.4	33.4	33.4	32.4	28-58
# 16	35.0	23.4	21.5	21.5	20.5	21.5	
# 30	27.0	17.3	15.5	15.5	14.5	13.8	
# 50	19.0	15.3	13.5	13.5	11.5	8.2	
# 100	9.0	9.2	11.4	11.4	10.4	6.4	
# 200	4.0	3.8	5.02	5.02	5.04	4.5	2-7

Table 6. Volumetrics of Mixes Containing RCA 1

RCA Percentage	0%	20%	40%	60%	80%	100%
Opt AC (%)	5.90	6.80	7.40	8.00	8.50	9.20
% Gmm N_{Ini}	85.35	85.29	84.78	84.82	84.86	85.15
% Gmm N_{Des}	96.00	95.83	95.86	96.05	96.00	95.82
VMA (%)	14.20	14.13	14.26	14.60	14.03	14.33
Air voids (%)	4.00	4.09	4.15	3.96	4.12	4.10
VFA (%)	71.90	71.78	70.63	73.05	69.10	69.17
Dust/ Asphalt ratio	0.92	0.85	1.05	1.12	1.10	0.62
Effective AC (%)	4.30	4.45	4.52	4.75	4.58	4.81
G_{mm}	2.583	2.460	2.388	2.313	2.260	2.219
G_{mb}	2.486	2.355	2.293	2.222	2.162	2.125
G_{se}	2.848	2.738	2.670	2.595	2.543	2.514

Table 7. Gradations of Blends of RCA 2

Sieve Size	Percent Passing for Different RCA Percentage					Gradation control Point
	20%	40%	60%	80%	100%	
3/4"	100.0	100.0	100.0	100.0	100.0	
1/2"	97.2	96.5	95.4	93.5	91.4	90-100
3/8"	87.5	81.2	80.3	74.7	72.3	90 max
# 4	60.3	50.7	49.4	43.8	43.4	
# 8	40.4	35.4	32.4	27.4	28.4	28-58
# 16	30.3	25.4	24.4	20.5	21.5	
# 30	22.5	20.3	19.6	17.4	19.3	
# 50	16.4	15.5	16.4	15.3	17.7	
# 100	7.4	11.1	13.4	11.7	17.4	
# 200	4.2	4.4	5.4	5.5	5.3	2-7

Table 8. Volumetrics of Mixes Containing RCA 2

RCA Percentage	0%	20%	40%	60%	80%	100%
Opt AC (%)	5.90	6.3	6.7	7	7.3	7.7
% Gmm N_{ini}	85.35	85.30	84.14	84.23	82.8	82.35
% Gmm N_{des}	96.00	96	96	96	96	96
VMA (%)	14.20	14.84	14.58	14.60	14.80	14.50
Air voids (%)	4.00	4.06	4.08	4.04	3.97	4.10
VFA (%)	71.90	73.4	72.57	72.60	72.97	72.41
Dust/ Asphalt Ratio	0.92	0.87	0.90	1.13	1.10	1.09
Effective AC (%)	4.30	4.60	4.07	4.40	4.20	4.57
G_{mm}	2.583	2.484	2.425	2.373	2.302	2.255
G_{mb}	2.486	2.388	2.424	2.376	2.205	2.159
G_{se}	2.848	2.749	2.687	2.632	2.559	2.504

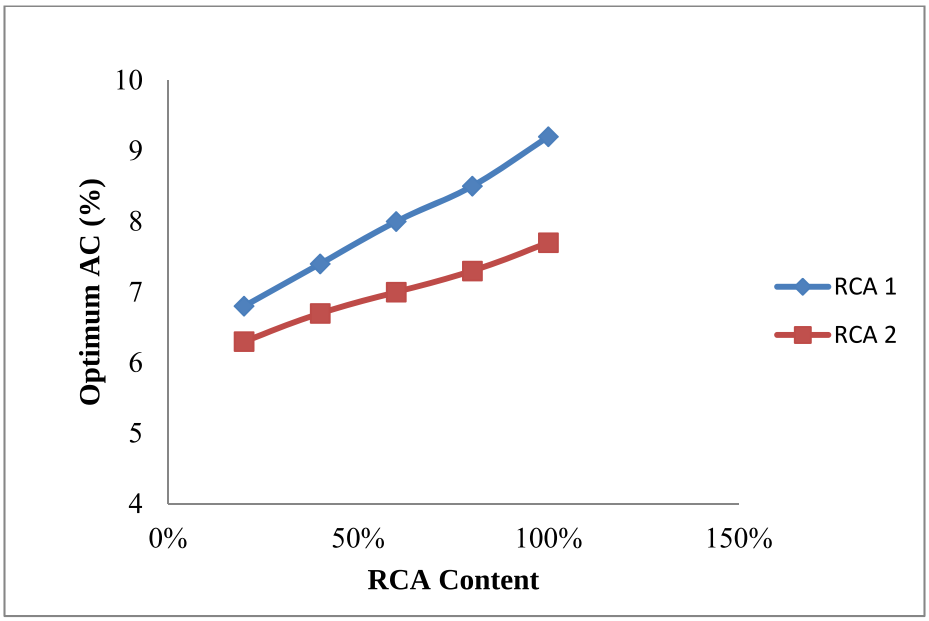

Figure 4. Optimum AC Content at Different RCA Percentages

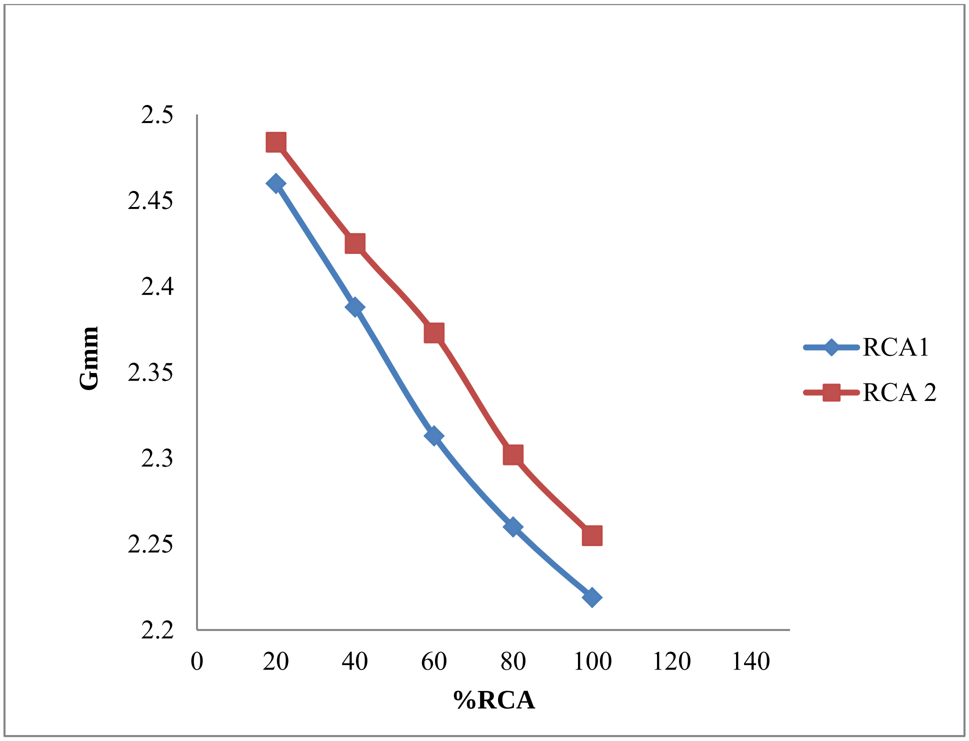

Figure 5. Theoretical Maximum Specific Gravity (G_{mm}) at Different RCA Percentages

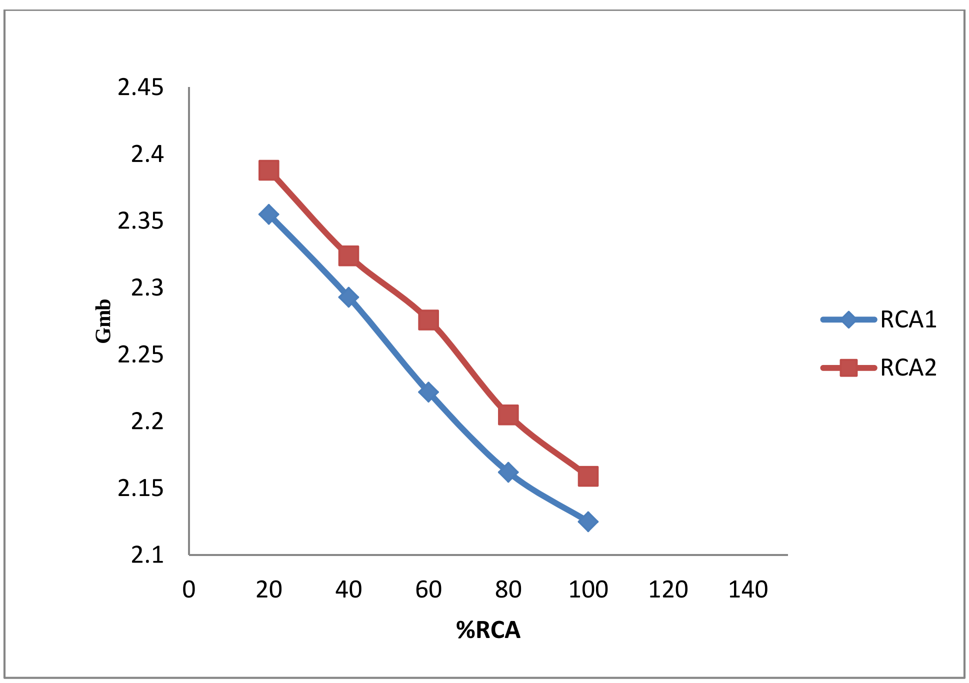

Figure 6. Bulk Specific Gravity (G_{mb}) at Different RCA Percentages

CHAPTER 3. LABORATORY EXPERIMENTS

3.1 INTRODUCTION

In asphalt pavement, fatigue cracking, rutting, thermal cracking and moisture damage are the primary distresses. HMA is designed in the laboratory to resist the distresses. However, currently, Superpave mix design used by most highway agencies is based on volumetrics. Direct measurements of performance of mixes are desired to evaluate the feasibility of using RCA as HMA aggregates. Many mechanical tests exist and have been used by researchers to characterize HMA. In this study, only the tests that have been demonstrated to correlate with field performance are selected. In addition, fundamental properties, such as dynamic modulus, are also characterized.

3.2 EXPERIMENTS

3.2.1 Dynamic Modulus

The dynamic modulus (|E*|) test evaluates the stiffness of HMA at different rates of loading and temperatures. In the Mechanistic-Empirical Pavement Design Guide (MEPDG), |E*| is a level 1 input for HMA material characterization [10]. It is an important HMA parameter for predicting the pavement performance. In the MEPDG, dynamic modulus of HMA is an input to determine critical response of an asphalt pavement in order to predict rutting, fatigue cracking and thermal cracking. HMA is a viscoelastic material and its behaviors are affected by temperature and rate of loading. Dynamic modulus is defined as the magnitude of the complex modulus. Figure 7 is a plot of the dynamic modulus in the complex plane. In Figure 7, dynamic modulus, |E*|, is shown as the magnitude of the elastic modulus and loss modulus. Also shown in Figure 7 is the phase angle φ between |E*| and elastic modulus. For an elastic material, the phase angle is zero and for a viscous material, the phase angle is 90°. Figure 8 shows the stress and strain curve in a dynamic modulus test. It also shows the phase lag between strain and stress. Dynamic modulus is calculated as the amplitude of the cyclic peak-to-peak stress divided by the cyclic recoverable strain as shown in Equation 3.1.

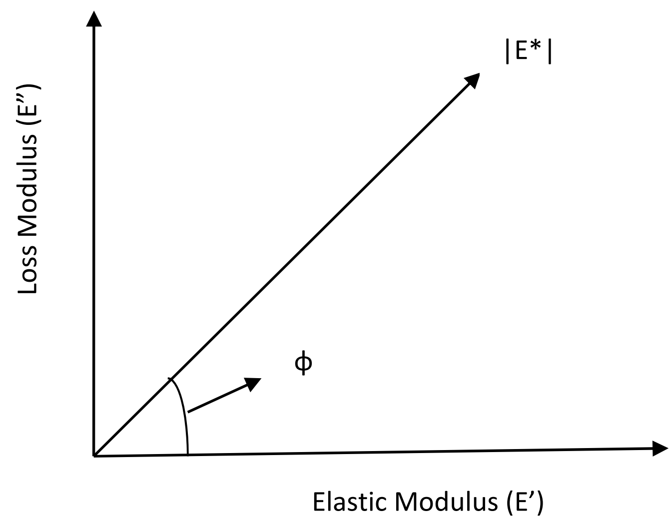

Figure 7. Dynamic Modulus in Complex Plane

$$|E^*| = \frac{\sigma_0}{\varepsilon_o} \tag{3.1}$$

where σ_0 represents peak to peak stress and ε_o represents the recoverable strain.

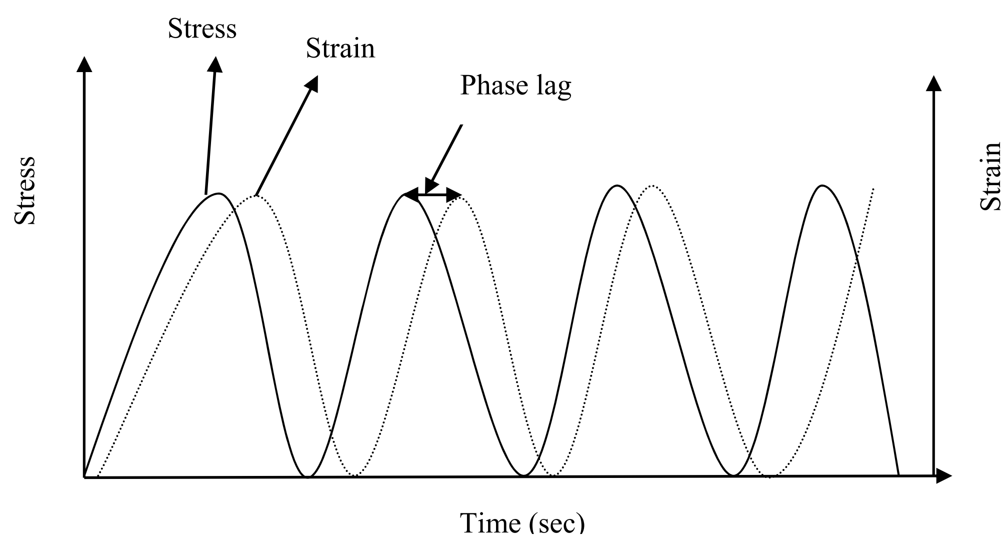

Figure 8. Typical Stress Strain Curve of Dynamic Modulus Test

AASTHO TP 79 test protocol was followed to conduct the dynamic modulus test. A Superpave gyratory compactor was used to prepare the test samples to a height of 6.7 inches (±0.2 inches). The air void levels were targeted at 8% (±0.5%). The compacted samples were then cored and cut to obtain the dynamic modulus test specimens (6 inches in height and 4 inches in diameter). The cored samples had a targeted air void level 7% (±0.5%). Numerous trials were performed to obtain this air void level prior to fabricating the test specimens. An asphalt mixture performance tester (AMPT) device was used to conduct the dynamic modulus test (Figure 9). The tests were performed at four different temperatures (44, 59, 70, and 98°F) and six different frequencies (0.01, 0.5, 15, 5, 10, and 25 Hz) at each temperature. Three replicates were tested for each mix.

Figure 9. Asphalt Mixture Performance Tester

3.2.2 Flow Number Test

The flow number test evaluates the high temperature rutting potential of HMA specimens. Flow number has been found to have good correlation with field rutting performance [11, 12]. Compared to the Hamburg wheel tracking test, the flow number test is easier to conduct. Figure 10 shows the loading pattern in a flow number test. Each cycle consists of 0.1 second loading following by 0.9 second rest period. After the specimen reaches the flow number, accelerated deformation starts. Figure 11 shows a typical axial strain response in a flow number test and Figure 12 shows the rate of change of axial strain. Flow number is defined as the maximum number of cycles to have a minimum rate of axial deformation or the beginning of the tertiary creep phase. The higher the number of cycles is needed to reach the flow number, the better the rutting performance is. The same specimens used for dynamic modulus tests which are non-destructive are used in flow number tests. The test temperature was 129°F. The deviatoric stress level was 87 psi and the contact stress was 1.4 psi. Three replicates were used for each mix. Figure 13 shows the test specimen after the flow number test.

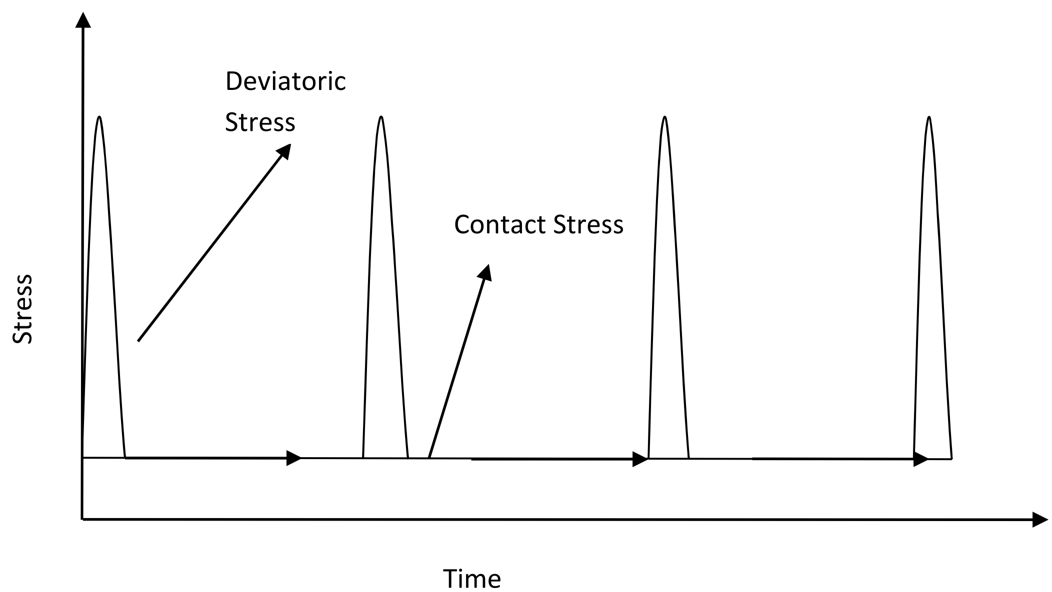

Figure 10. Haversine Loading for Flow Number

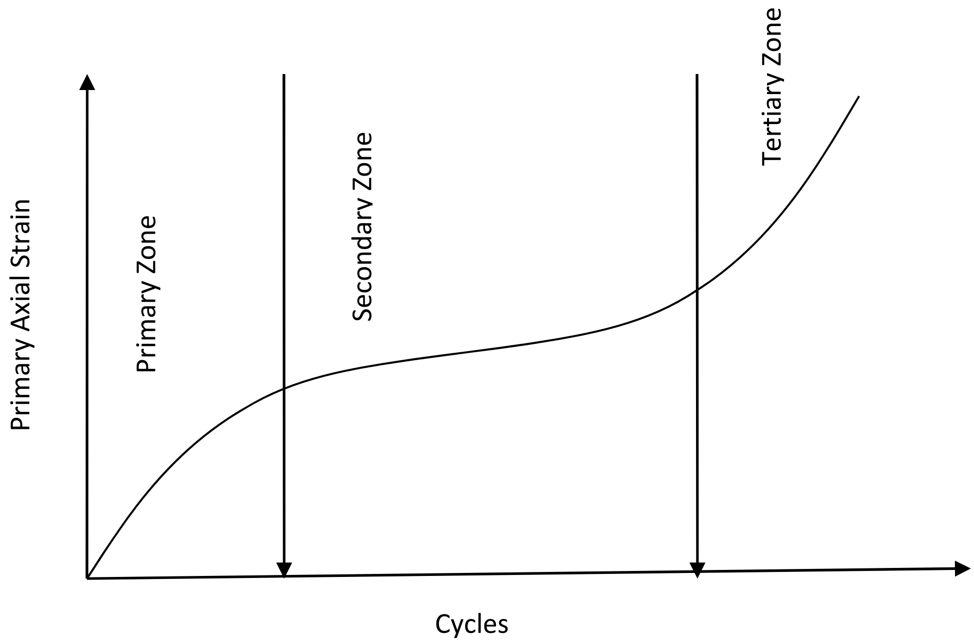

Figure 11. Primary Strain Accumulations in Flow Number Test

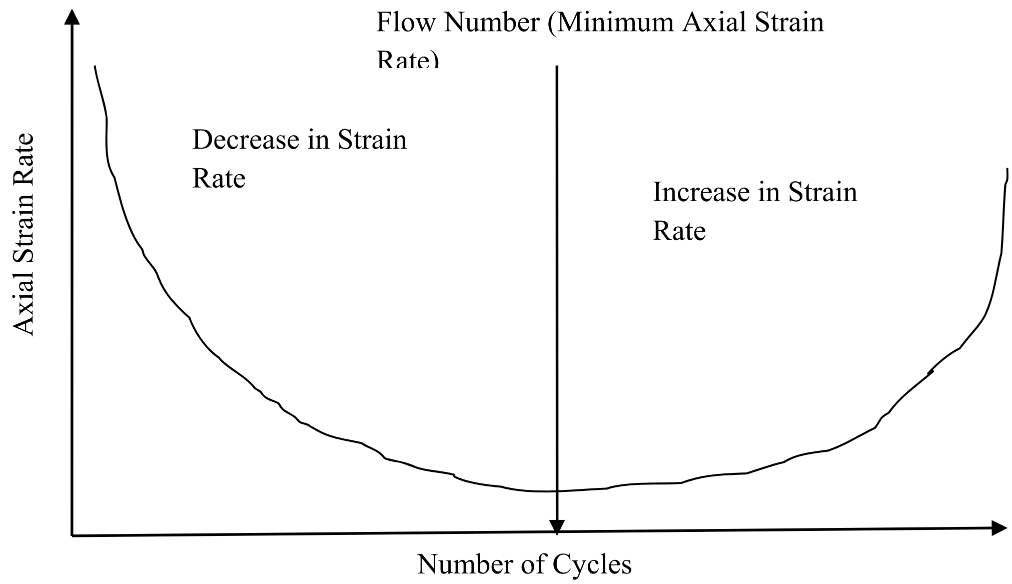

Figure 12. Axial Strain Rate with Number of Cycles

Figure 13. Test Specimen after Flow Number Test

3.2.3 Indirect Tensile Test (Fatigue and Thermal Cracking)

Fatigue and thermal cracking distresses are related to the tensile behavior of asphalt mixes. The indirect (IDT) tensile test evaluates the tensile behavior of HMA mixes. In an IDT test, specimens are loaded diametrically to generate the tensile stress along the vertical diameter. The peak stress represents tensile strength of the specimen. The fracture energy obtained from the IDT strength test at intermediate temperature was found to correlate with field fatigue performance [13]. Figure 14 shows a typical plot of IDT test results. The area under the curve of a stress versus strain plot, up to the peak point, is the fracture energy (FE) of the mix. Fracture energy was also found to be able to predict the thermal cracking behavior of asphalt concrete [14]. High fracture energy indicates high resistance to fatigue and thermal cracking. AASTHO T 322 test standard was followed to conduct the IDT test [15]. Test samples were compacted by the Superpave gyratory compactor to a height of 4.5 inches. The air void levels of the gyratory compacted samples were maintained at 6% (±0.5%). The gyratory compacted samples were then cored and cut for obtaining IDT test specimens with a diameter of 4 inches and a height of 1.5 inches. The air void levels of the cored test specimens were maintained at 4% (±0.5%). Numerous trials were performed to obtain this air void level prior to fabricating actual test specimens. To measure the horizontal and vertical deformations, studs were glued on both sides of test specimen. Figure 15 shows the gluing device used for mounting studs. The gauge length of the mounting studs was kept at 2 inches to minimize the effect of large size particles [14]. A Geotechnical Consulting and Testing Systems (GCTS) machine was used to conduct the IDT tests. Figure 16 shows the test apparatus set up for running the IDT test. Three replicates were used for each mix.

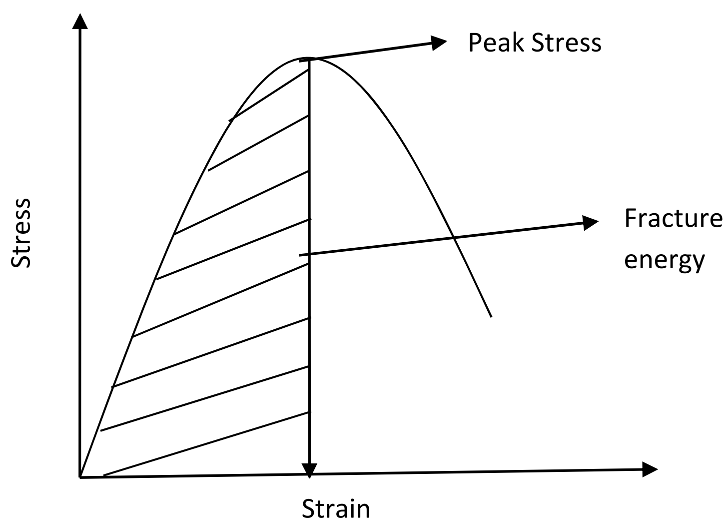

Figure 14. IDT Stress and Strain

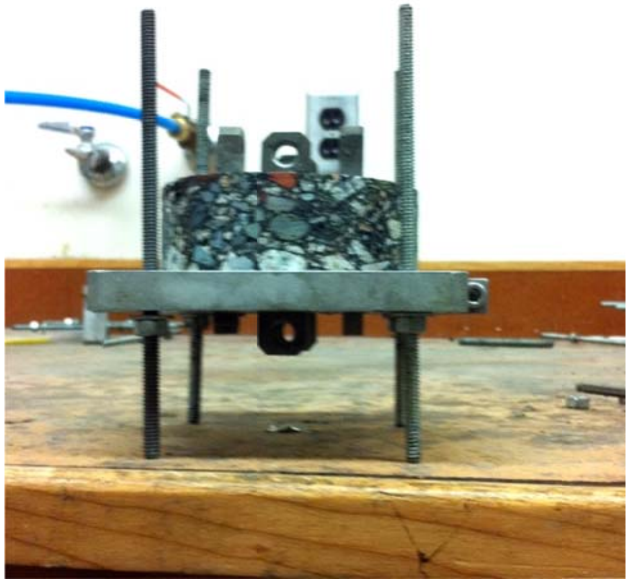

Figure 15. Stud Gluing Device for IDT Test

Figure 16. IDT Test Setup

To determine the IDT tensile strength and fracture energy, the test specimens were loaded under a constant rate of movement of piston (2 inches/minute) for both the fatigue and thermal cracking tests. Fatigue cracking of HMA generally occurs at intermediate temperatures. Therefore, the IDT tests for fatigue were performed at room temperature. Thermal cracking of HMA occurs at low temperatures. Therefore, the IDT tests for thermal cracking were performed at 14°F. The low temperature was selected according to AASTHO T 322 standard for a PG 58-28 binder. For the IDT at low temperatuers, test specimens were kept inside the universal testing chamber at 14°F for overnight prior to running the test. Test specimens were loaded until a clear peak stress of failure was seen for both fatigue and thermal cracking. Test data were then analyzed to calculate the tensile strength and fracture energy of the mix. The stress/strain distribution along the horizontal diameter is not uniform. However, the tensile failure occurs along the vertical diameter. The horizontal displacement measured by the linear differential voltage transducer (LVDTs) needs to be converted to the tensile strain at the center of specimen. To determine the fracture energy, Equation 3.2 was used to determine the center point strain based [13].

$$\varepsilon_{x=0} = U\frac{a+b\nu}{c+d\nu} \qquad (3.2)$$

Where $\varepsilon_{x=0}$ = Center point strain,

a, b, c, d = Coefficient related to specimen diameter and gauge length,

U= Horizontal displacement,

ν = Poisson's Ratio.

Table 9 shows the coefficients related to the diameter and gauge length of the test specimen

Table 9. Coefficients in Equation 3.2 (IDT Test)

Diameter	Gauge length	a	b	c	d
4 inches	2 inches	12.4	37.7	0.471	1.57

Poisson's ratio, ν, is calculated based on Equation 3.3.

$$\nu = -\frac{a_1 U(t) + b_1 V(t)}{a_2 U(t) + b_2 V(t)} \tag{3.3}$$

where a_1, b_1, a_2, and b_2 are coefficients related to specimen diameter and gauge length. U (t) and V (t) are the displacements of LVDT at time t. Table 10 shows the coefficient related to diameter and gauge length in this study for calculation of Poisson's ratio [13].

Table 10. Coefficient in Equation 3.3

Diameter	Gauge length	a_1	b_1	a_2	b_2
4 inches	2 inches	4.580	1.316	1	3.341

Three replicates were tested for each mix. Figures 17 and 18 show the test specimens after the IDT fatigue and thermal tests, respectively.

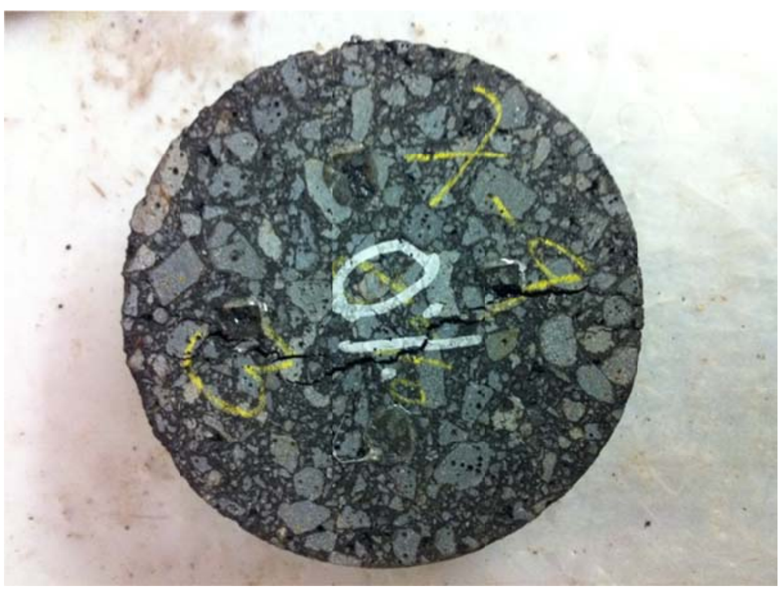

Figure 17. Test Specimens after IDT Fatigue Test

Figure 18. Test Specimens after IDT Test at Low Temperature

3.2.4 Moisture Sensitivity Test

The moisture sensitivity test evaluates the susceptibility of a mix to moisture damage. Intrusion of moisture inside HMA decreases the durability of mixes by lowering the bond strength between the aggregate and asphalt binder. This test evaluates the moisture susceptibility of HMA samples by comparing the tensile strengths of conditioned and unconditioned samples to determine the tensile strength ratio (TSR). Conditioned samples are subjected to freeze and thaw cycles whereas unconditioned samples are stored at room temperature. WSDOT T718 test procedure was followed to conduct this test [16]. The loading rate is 0.065 inch per minute. To prepare the test specimens, samples were compacted to a height of 3.7 inches. The air void level was maintained at 4% (±0.5%). A total of six samples were prepared for each mix. For the first two samples, no anti-stripping additive was added and for the other 4 samples, anti-stripping additive was added at 0.25, 0.5, 0.75, and 1% by the weight of asphalt binder, respectively. One of the samples with no anti-stripping additive was kept as reference sample (i.e. unconditioned) and stored at room temperature. The other 5 samples were first vacuum-saturated to a degree of saturation between 60-80 percent. Freezing and thawing cycles were carried according to the WSDOT 718 test standard. Figure 19 shows the test apparatus set up for running the moisture susceptibility test. The samples were allowed to fail along the diametrical loading (indirect tensile loading). Test specimens were loaded to the peak load of failure and the tensile strengths of each specimen was calculated. The minimum TSR value for a HMA mix to pass the WSDOT specification is 80%.

Figure 19. Moisture Damage Test

CHAPTER 4. PERFORMANCE TEST RESULTS

The experimental results are analyzed to evaluate the effects of RCA on performance of HMA, in terms of modulus, fatigue, rutting, thermal cracking and moisture susceptibility.

4.1 DYNAMIC MODULUS TEST

Tables 11 and 12 show the dynamic modulus test results of the mixes containing RCA and control HMA mixes. Asphalt concrete is a thermorheologically simple material, exhibiting viscoelastic behaviors. Based on the time temperature superposition principle, HMA stiffness (E*) can be obtained over a large range of temperatures or frequencies by generating a master curve. To generate the master curve, the dynamic modulus at each temperature is horizontally shifted to the reference temperature. The master curve is then fitted to a sigmoidal function as shown in Equation 4.1. Figure 20 shows the typical plot showing the dynamic modulus of a test specimen before and after shifting to the reference temperature.

$$Log\,|E^*| = a + \frac{b}{1 + \frac{1}{\exp^{d + e\log f}}} \tag{4.1}$$

where f is frequency of loading, and a, b, d, and e are regression coefficients.

Figures 21 and 22 show the dynamic modulus master curves of the mixes containing RCA 1 and 2 at the reference temperature of 21°C, respectively. The results indicated that increasing the percentage of RCA in the mix decreased the dynamic modulus (stiffness) of the mix. This finding agrees with that of Paranavithana et al. [4] and Beale et al. [7]. This might be due to the high asphalt content in mixes containing RCA.

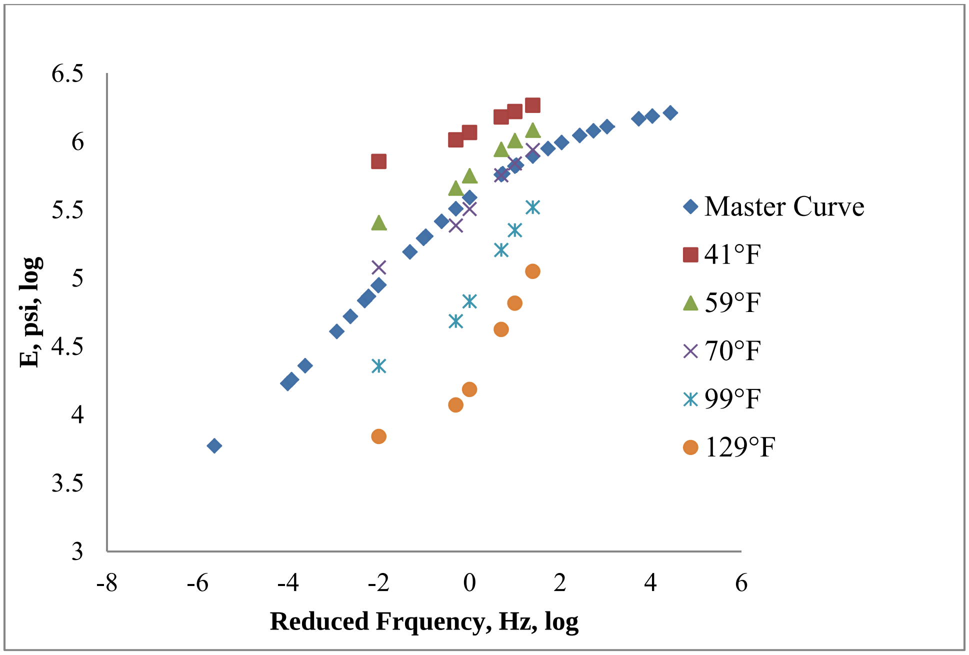

Figure 20. Dynamic Modulus before Horizontally Shifting to Master Curve

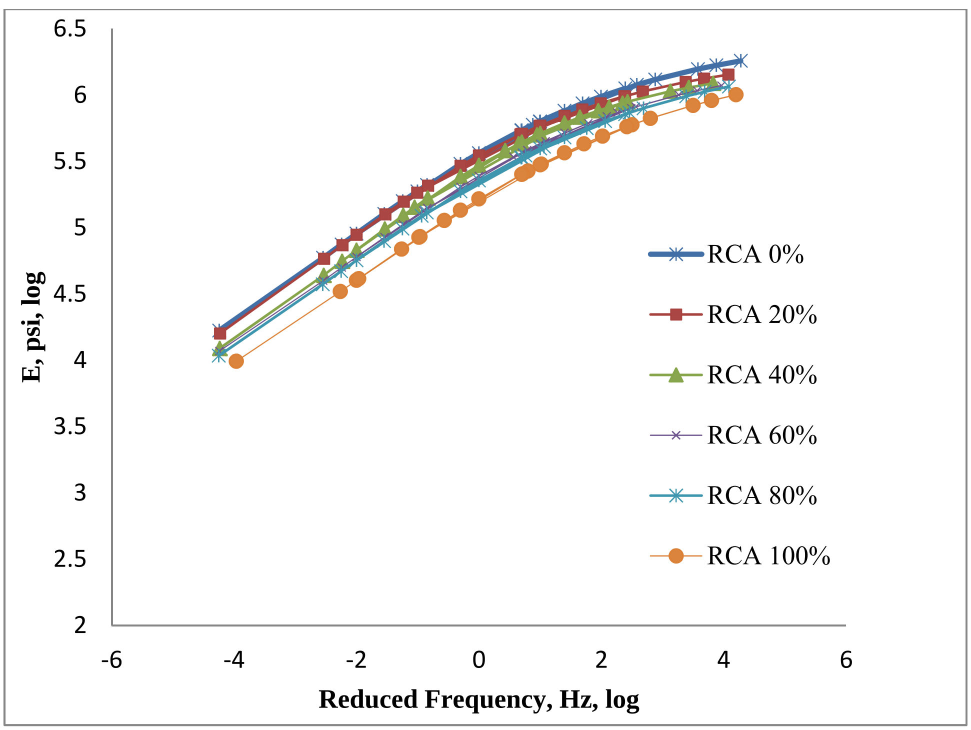

Figure 21. Dynamic Modulus Master Curve of Mixes Containing RCA 1

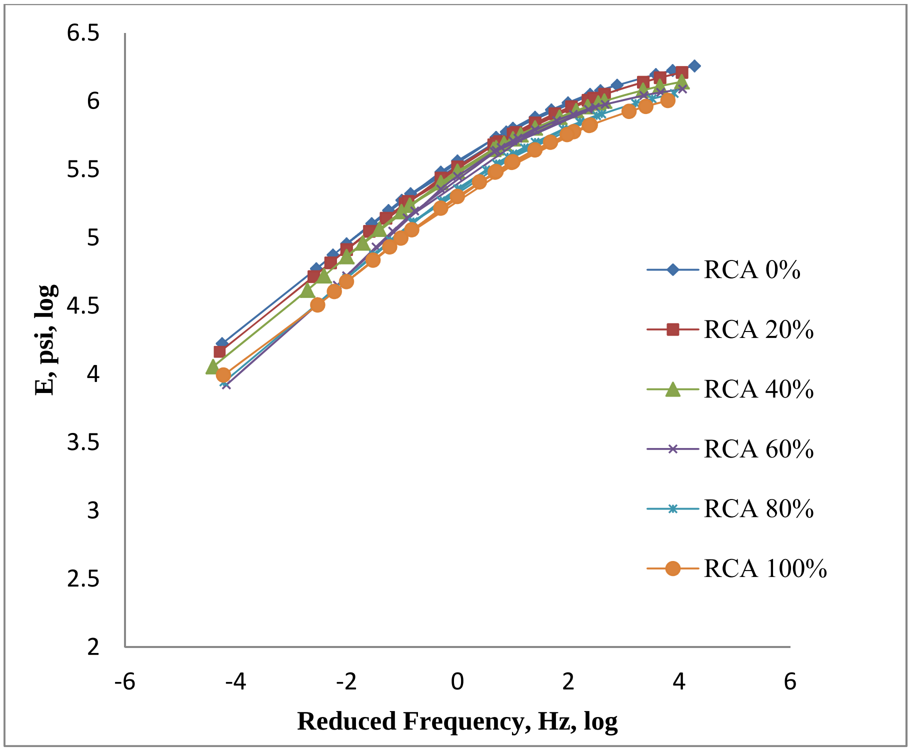

Figure 22. Dynamic Modulus Master Curve of Mixes Containing RCA 2

Table 11. Dynamic Modulus Test Results for Mixes Containing RCA 1

Temperature °F	Frequency	Average |E*| (ksi) for Different RCA Percentage					
		100%	**80%**	**60%**	**40%**	**20%**	**0%**
44.6	25	1083.6	1249.3	1319.4	1425.5	1649.7	2006.9
	10	937.9	1097.4	1144.2	1230.3	1439.0	1755.4
	5	827.4	976.7	1014.0	1089.8	1282.2	1563.7
	1	590.6	704.3	727.3	779.7	950.9	1165.4
	0.5	498.8	604.5	615.5	663.4	817.8	1003.4
	0.01	312.8	387.6	389.3	423.4	547.5	672.7
49	25	646.4	824.8	894.1	959.0	1082.7	1216.0
	10	515.3	673.7	730.7	786.8	889.6	996.2
	5	423.3	563.6	607.0	669.9	757.9	862.9
	1	254.2	348.4	380.2	419.3	491.6	541.6
	0.5	202.0	274.2	304.2	337.9	405.9	418.5
	0.01	106.1	150.7	165.6	180.8	231.0	241.1
69.8	25	426.4	545.9	589.0	684.3	771.4	867.4
	10	326.7	424.7	462.6	531.9	602.9	692.1
	5	259.0	342.6	377.6	434.4	505.0	561.4
	1	137.5	189.1	206.6	241.9	294.8	312.8
	0.5	102.8	145.3	155.6	185.6	227.9	242.0
	0.01	49.6	71.3	75.0	92.1	115.7	105.1
98.6	25	151.4	181.3	194.2	211.8	263.3	283.9
	10	103.5	119.5	128.3	140.7	178.6	193.4
	5	74.0	84.8	90.6	100.9	129.3	131.5
	1	32.2	35.8	38.3	44.1	58.0	56.7
	0.5	23.3	25.7	27.4	32.0	42.1	43.6
	0.01	10.6	12.0	13.1	15.2	20.3	18.4

Table 12. Dynamic Modulus of Mixes Containing RCA 2

Temperature °C	Frequency	Average |E*|(ksi) for Different RCA %					
		100%	80%	60%	40%	20%	0%
44.6	25	1115.3	1325.0	1473.9	1555.7	1766.5	2006.9
	10	952.2	1149.1	1280.8	1353.1	1561.1	1755.4
	5	839.6	1015.6	1140.9	1211.5	1389.5	1563.7
	1	584.5	728.2	834.8	900.0	998.2	1165.4
	0.5	489.8	619.4	713.3	773.6	848.4	1003.4
	0.01	299.4	385.8	456.6	500.7	547.4	672.7
49	25	735.2	924.1	1069.8	1103.6	1160.0	1216.0
	10	596.0	748.1	883.6	911.3	954.0	996.2
	5	491.6	637.4	740.7	759.8	798.4	862.9
	1	300.3	393.5	460.8	477.5	498.7	541.6
	0.5	235.9	314.4	370.0	386.7	406.1	418.5
	0.01	126.7	167.2	200.2	210.4	221.1	241.1
69.8	25	507.9	620.6	727.3	739.8	785.6	867.4
	10	387.6	476.8	553.6	567.0	622.8	692.1
	5	311.5	380.5	452.4	465.2	501.8	561.4
	1	169.2	205.9	249.5	256.9	278.5	312.8
	0.5	125.7	153.8	185.6	191.3	210.7	242.0
	0.01	59.2	72.5	88.6	91.8	101.6	105.1
98.6	25	163.6	198.9	202.1	210.0	257.5	283.9
	10	106.7	131.7	133.1	140.1	171.0	193.4
	5	74.5	91.5	93.7	99.0	120.2	131.5
	1	29.3	37.4	38.0	40.6	49.9	56.7
	0.5	20.7	26.5	26.7	28.6	34.9	43.6
	0.01	11.3	11.7	12.3	12.6	16.4	18.4

4.2 FLOW NUMBER TEST

Table 13 shows the average flow number test results for RCA 1 and 2. The test results indicated that the flow number significantly decreased with an increase in RCA percentage, resulting in reduced resistance to rutting. Figures 23 and 24 show the relationship between flow number and RCA percentage for RCA 1 and 2, respectively. The reduced resistance to rutting which results from the use of RCA could be due to the increased asphalt content in the mix. According to the Superpave mix design, the absorbed asphalt by the aggregate is not effective asphalt and does not affect the mix's behavior. However, at elevated temperature, such as that used for flow number testing, the expansion of asphalt could result in an increased amount of "effective binder" which would lead to a series of issues, such as susceptibility to rutting and bleeding in hot weather. It is to be noted that the mixes containing RCA meet all the volumetric requirements and asphalt/aggregate specifications. The flow number test results indicated the deficiency of relying solely on the volumetrics. Performance-based tests are needed to supplement the volumetric design.

Table 13. Flow Numbers of Mixes Containing RCA 1 and 2

RCA%	Flow Number	
	RCA 1	RCA 2
0%	185	185
20%	135	140
40%	87	83
60%	71	58
80%	39	34
100%	24	27

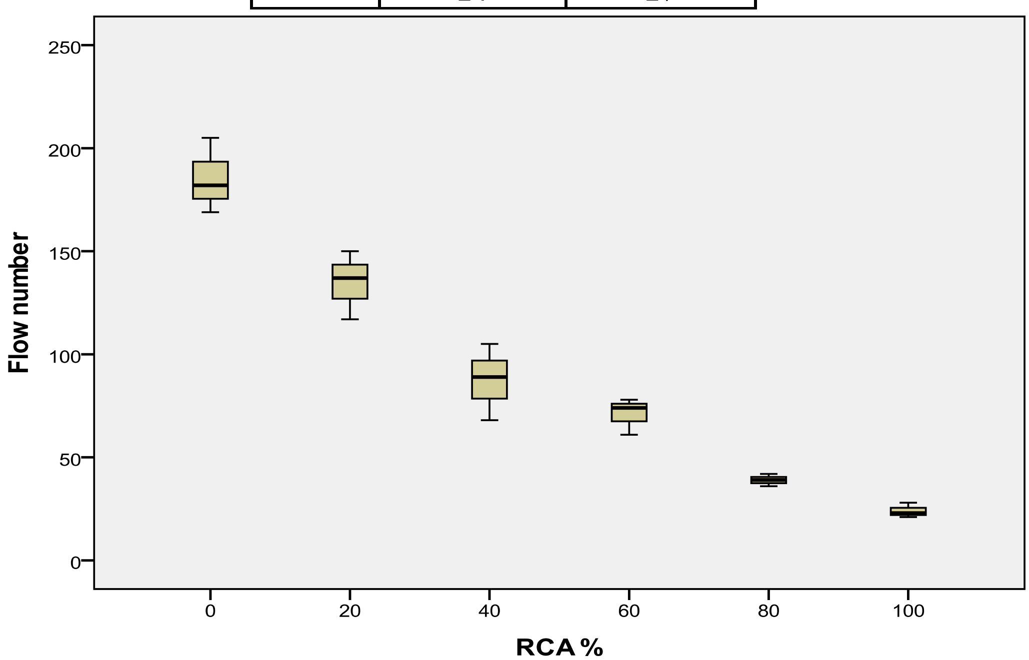

Figure 23. Flow Number of Mixes Containing RCA 1

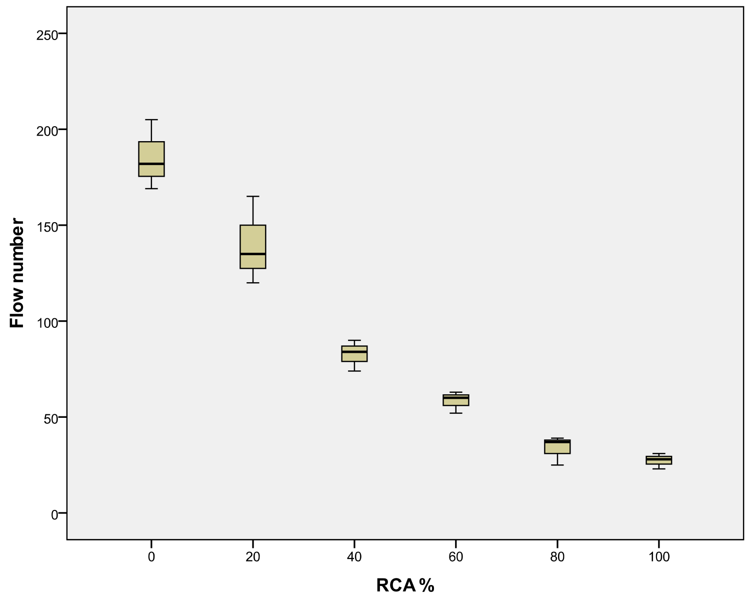

Figure 24. Flow Number of Mixes Containing RCA 2

4.3 INDIRECT TENSILE TEST FOR FATIGUE

Tensile Strength

The IDT tensile strength of mixes are shown in Table 14. Figures 25 and 26 present tensile strength of RCA at different RCA percentages. Test results show that the tensile strength of the mix decreased with an increase in RCA percentage. The lowered tensile strength might be due to the increased asphalt content and/or crushing of RCA. It is noted that crushed RCA were observed at the fractured faces of specimens.

Table 14. Tensile Strength of Mixes Containing RCA 1 and 2

RCA%	Source 1	Source 2
	Tensile Strength (psi)	Tensile Strength (psi)
0%	157.6	157.6
20%	143.0	124.1
40%	130.8	111.9
60%	123.0	104.4
80%	117.8	88.5
100%	110.6	79.1

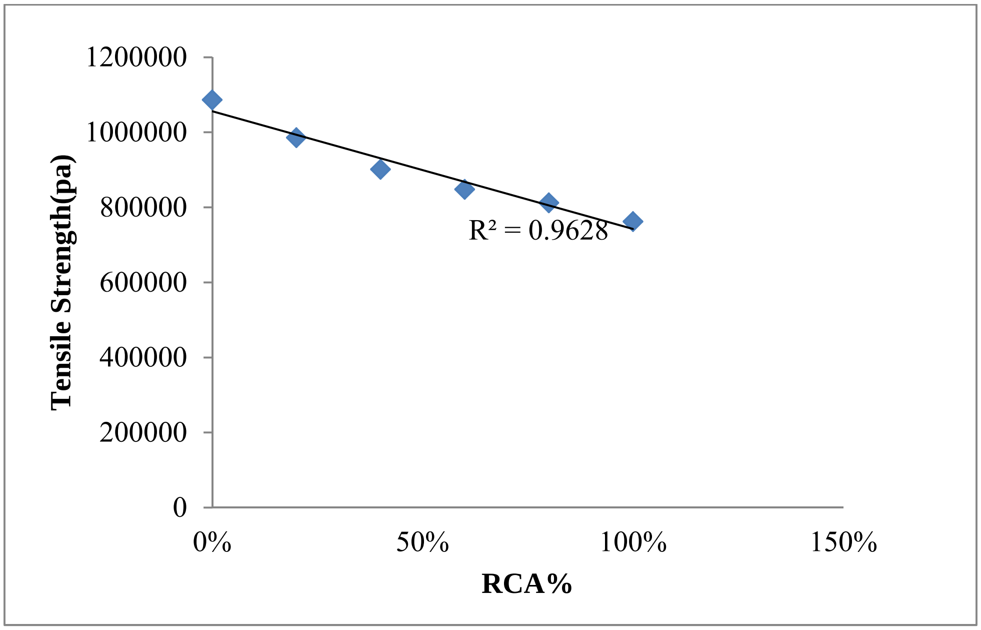

Figure 25. Fatigue Tensile Strength of RCA Source 1 at Different RCA%

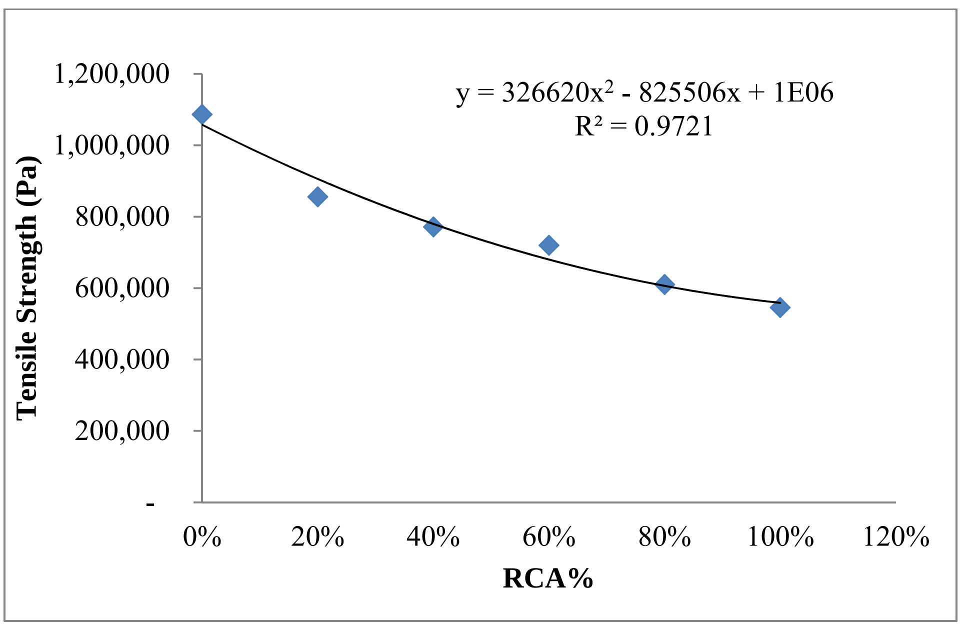

Figure 26 Fatigue Tensile Strength of RCA Source 2 at Different RCA %

Fracture Energy

The IDT fracture energy of mixes is shown in Table 15. Figure 27 indicates that for RCA 1, there is no clear relationship between fracture energy and RCA percentage. The 80% RCA mix shows the largest fracture energy and 20% RCA mix shows the lowest fracture energy. For mixes containing RCA 2, Figure 28 indicates that with an increase in RCA percentage, the fracture energy decreased, resulting in a reduced resistance to fatigue. The lowered fracture energy for mixes containing RCA 2 might be due to the increased asphalt content and/or crushing of RCA. However, it is unknown why similar phenomenon did not occur to mixes containing RCA 1.

Table 15. Fracture Energy of Mixes at Intermediate Temperature

RCA%	Source 1 Fracture Energy (psi)	Source 2 Fracture Energy (psi)
0%	1.38	1.38
20%	1.13	1.20
40%	1.27	0.82
60%	1.23	0.66
80%	1.56	0.53
100%	1.31	0.42

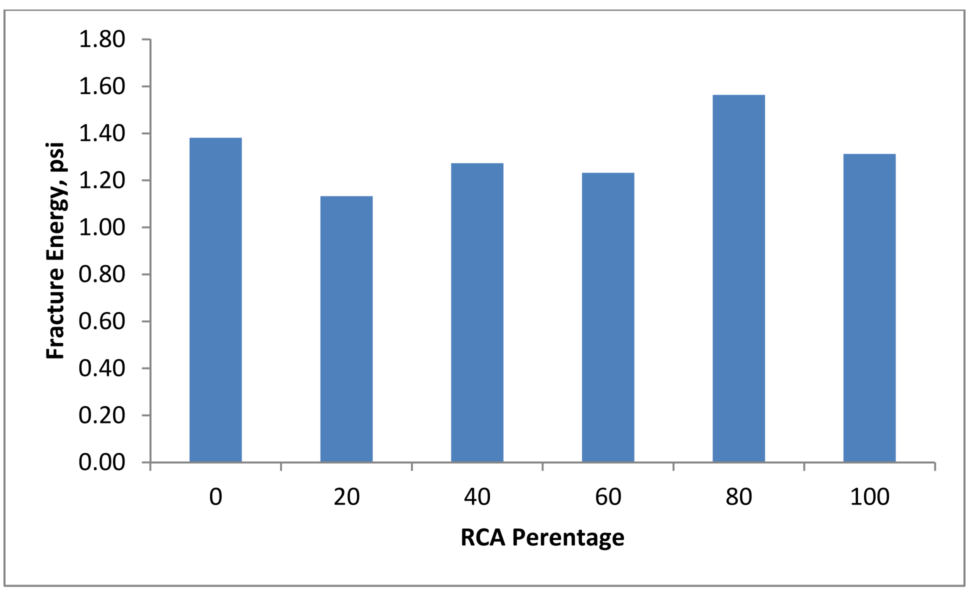

Figure 27. Fracture Energy of Mixes Containing RCA 1

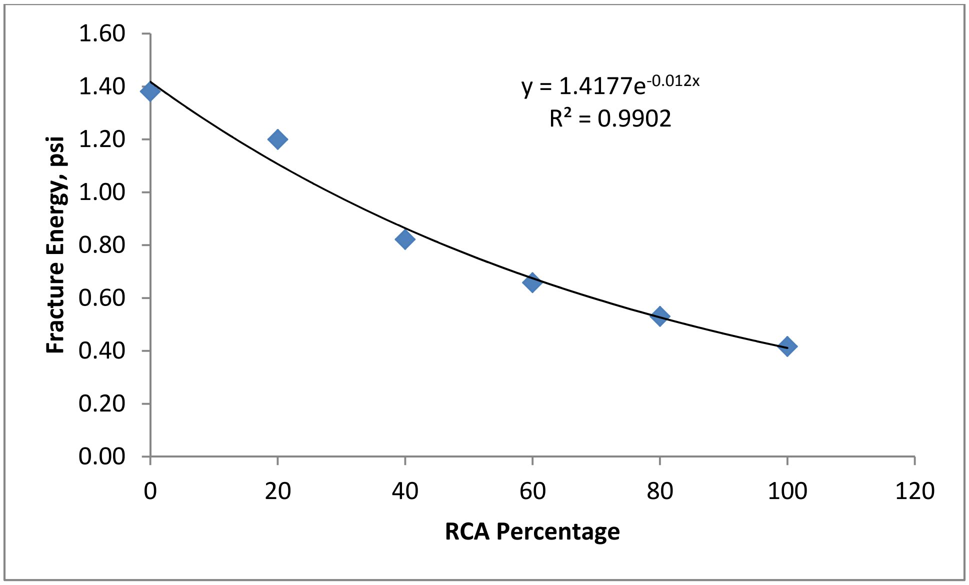

Figure 28. Fracture Energy of Mixes Containing RCA 2

4.4 INDIRECT TENSILE TEST FOR THERMAL CRACKING

<u>IDT Tensile Strength</u>

The tensile strength of mixes at low temperature is shown in Table 16. Figures 29 and 30 show the relationships between the IDT tensile strength and RCA percentage. Test results showed that for both mixes containing RCA 1 and 2, increasing the RCA percentage decreased the tensile strength of the mixes. For mixes with RCA, RCA aggregates were found to be broken at the failure faces. It is believed that the increase asphalt content and crushing of RCA led to lowered tensile strength.

Table 16. Tensile Strength of Mixes Containing RCA 1 and 2

RCA%	Source 1 Tensile Strength (psi)	Source 2 Tensile Strength (psi)
0%	869.8	869.8
20%	859.9	866.0
40%	791.9	726.7
60%	745.9	676.8
80%	656.4	708.5
100%	644.3	598.5

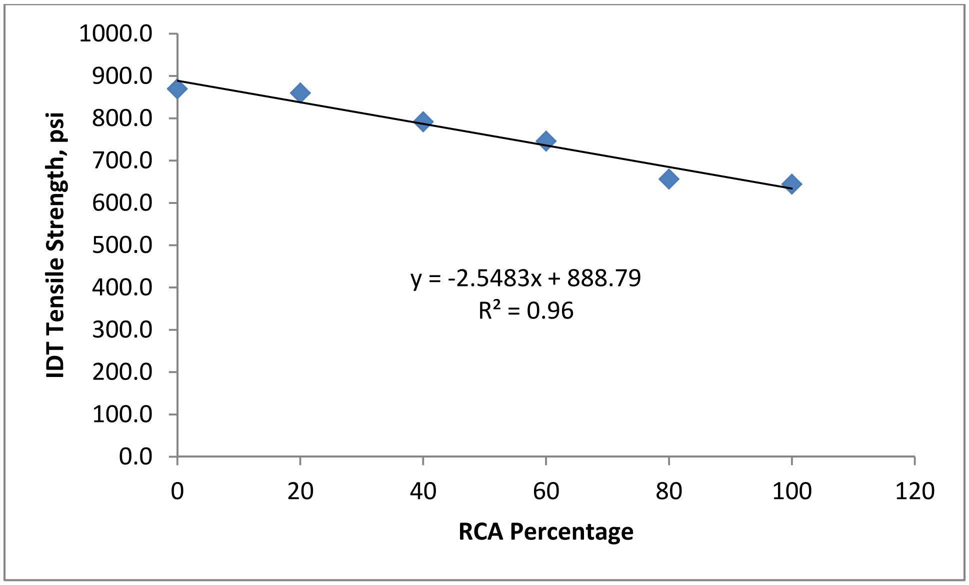

Figure 29. Low Temperature Tensile Strength of Mixes Containing RCA 1

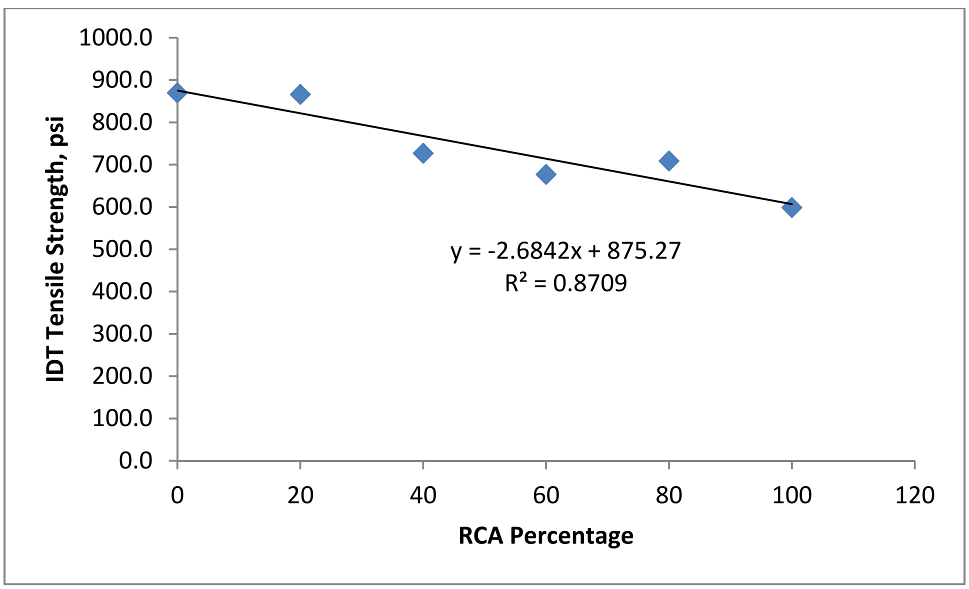

Figure 30. Low Temperature Tensile Strength of Mixes Containing RCA 2

<u>Low Temperature Fracture Energy</u>

The low temperature fracture energy is shown in Table 17. Figures 31 and 32 present the relationships between fracture energy and RCA percentage for mixes containing RCA 1 and

RCA 2, respectively. Similar to the fracture energy at intermediate temperature, there was no correlation between the fracture and RCA 1 percentage, whereas for mixes containing RCA 2, increasing RCA 2 percentage decreased the fracture energy, which resulted in reduced resistance to thermal cracking. However, the reason that there is no such relationship for mixes containing RCA 1 remains unknown.

Table 17. Low Temperature Fracture Energy of Mixes Containing RCA 1 and 2

RCA Percentage	RCA 1 Fracture Energy (psi)	RCA 2 Fracture Energy (psi)
0%	3.3	2.4
20%	3.2	2.0
40%	1.6	2.0
60%	2.9	1.9
80%	2.8	1.7
100%	3.6	1.0

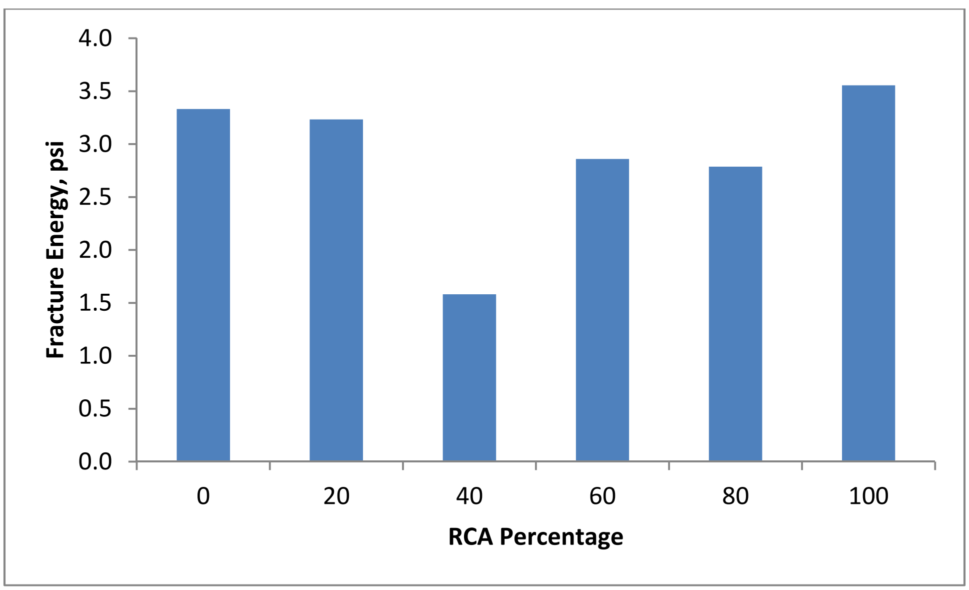

Figure 31. Low Temperature Fracture Energy of Mixes Containing RCA 1

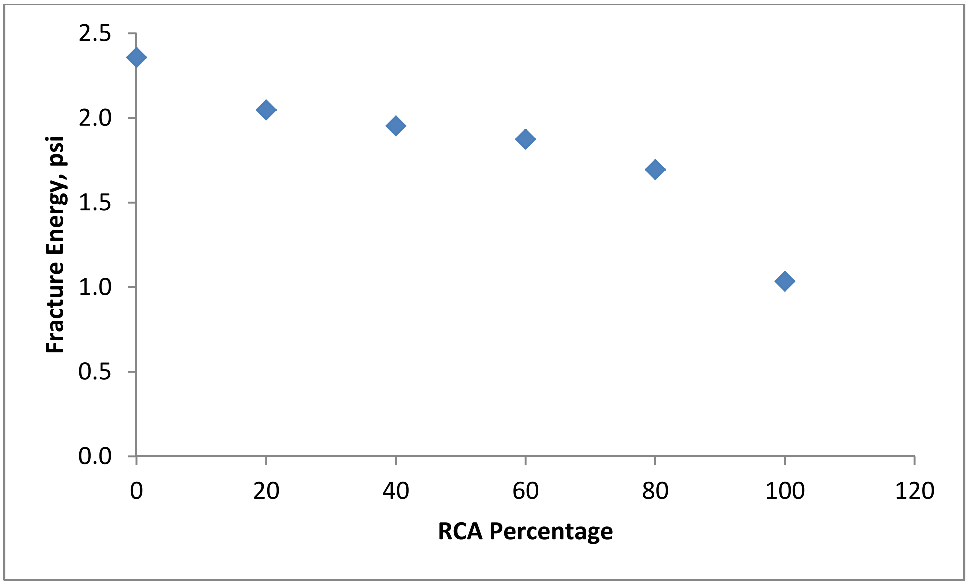

Figure 32. Low Temperature Fracture Energy of Mixes Containing RCA 2

4.5 MOISTURE SENSITIVITY

The test results on moisture susceptibility of mixes containing RCA are shown in Tables 18 and 19, respectively. The test results showed that the mixes containing 100% and 80% RCA 1 mixes with 0% anti-stripping additive did not pass the minimum TSR requirement of 80% specified by WSDOT. These mixes needed 0.25% anti-stripping additive to pass the TSR of 80%. For both RCA sources 1 and 2, it was found that increasing the percentage of anti-stripping agent in the mix increased the TSR values, as expected. Figures 33 and 34 show the TSR value at 0% anti-stripping additive for mixes containing RCA 1 and 2, respectively. Overall, the test results indicate that the addition of RCA increased the mix moisture susceptibility. Visual observation of the RCA mixes showed no stripping of asphalt from the aggregate after the samples failed. However, crushing of the recycled concrete aggregate was observed. It is believed that the increased asphalt content and/or crushing of RCA caused the reduced TSR values.

Table 18. TSR of Mixes Containing RCA 1

Anti-strip Content	RCA Percentage					
	100%	80%	60%	40%	20%	0%
0%	0.76	0.77	0.80	0.82	0.87	0.88
0.25%	0.81	0.83	0.81	0.86	0.89	0.89
0.5%	0.93	0.97	0.90	0.93	0.90	0.92
0.75%	0.96	0.96	1.00	0.97	0.93	0.97
1.0%	0.99	1.01	1.03	1.00	1.01	1.01

Table 19. TSR of Mixes Containing RCA 2

Anti-strip Content	RCA Percentage					
	100%	80%	60%	40%	20%	0%
0%	0.80	0.81	0.81	0.82	0.84	0.88
0.25%	0.82	0.83	0.82	0.82	0.84	0.89
0.5%	0.84	0.85	0.83	0.83	0.88	0.92
0.75%	0.85	0.90	0.88	0.86	0.92	0.97
1.0%	0.85	0.91	0.90	0.90	0.97	1.01

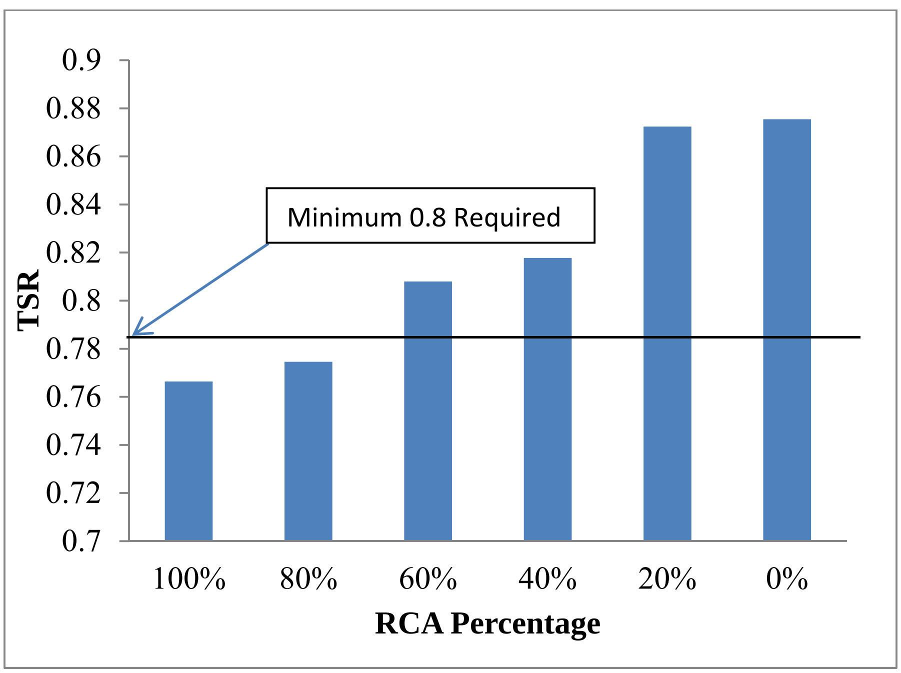

Figure 33. TSR Value Mixes Containing RCA 1 (No Anti-Stripping Additive)

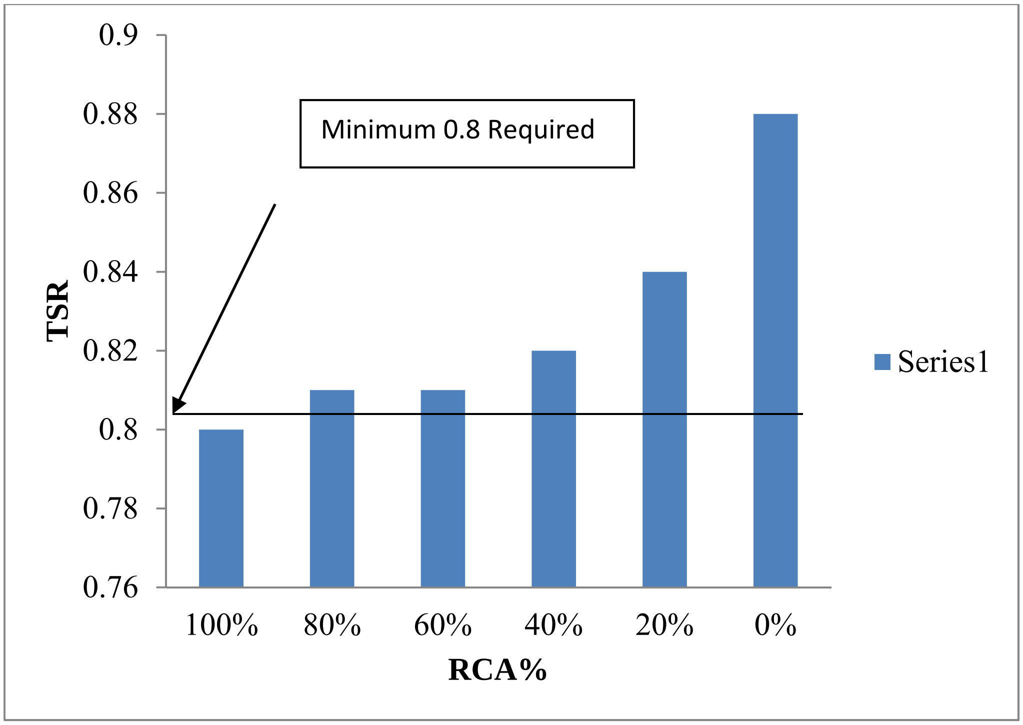

Figure 33. TSR Value Mixes Containing RCA 2 (No Anti-Stripping Additive)

CHAPTER 5. CONCLUSIONS AND RECOMMENDATIONS

This study evaluated the effects of replacing HMA aggregates with recycled concrete aggregates. Two sources of RCA were procured and used in this study. The evaluation consisted of two aspects: effects of RCA on mix design and effects of RCA on performance of HMA. Based on the experimental evaluation, the following conclusions could be observed:

(1) The two sources of RCA used in this study are highly absorptive and the porous structure leads to low specific gravity, especially for fine RCA.

(2) The two sources of RCA meet most of requirements on the source and consensus properties for HMA aggregates, except for the WSDOT degradation value. Blending RCA with at least 20% virgin aggregates can lead to combined aggregates which pass the WSDOT degradation tests.

(3) The increase of RCA percentage leads to an increase of asphalt content, due to the high absorption of RCA.

(4) The use of RCA significantly reduced the flow number, tensile strength at both intermediate and low temperatures, fracture energy at intermediate and low temperature, and TSR, resulting in reduced resistance to rutting, fatigue, thermal cracking and moisture damage.

(5) Superpave volumetric design method can not capture the performance issues of mixes and performance-based tests are needed to supplement the volumetric design to ensure good performance of mixes.

The following recommendations can be made:

(1) More RCA sources need to be included.

(2) It should be cautious to use RCA to replace virgin aggregates, because the performance of HMA could be compromised. In addition, the high asphalt content would increase the costs.

(3) The current volumetrics-based mix design method should be supplemented with criteria based on performance tests.

(4) The criteria based on performance tests needs to be developed and implemented.

(5) The concept of effective binder content in current mix design should be investigated and/or validated.

CHAPTER 6. REFERENCES

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