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Evaluation of the Effectiveness of Soylei Recycling Agent (RA) in High RAP Asphalt Mixtures in New York City

Final Report

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INTRODUCTION

A research study was initiated to evaluate the effectiveness of the Soylei recycling agent (RA) in the utilization of higher recycled asphalt pavement (RAP) content in hot mix asphalt. The RA was used in a 45% and 50% RAP asphalt mixture produced and placed in Staten Island, NY. The RA modified asphalt mixtures were compared to a baseline 40% RAP asphalt mixture used as a “Control” mix. The New York City Department of Transportation (NYCDOT) requires all asphalt mixtures to be produced with 40% RAP in an effort to recycle greater amounts of millings from rehabilitated asphalt pavements within the New York City jurisdiction.

The asphalt mixtures were designed and produced by City Asphalt in Staten Island, NY. The City Asphalt plant is a Gencor drum plant set up for 300 tons per hour (Figure 1). The RAP at the facility is fractionated and covered to minimize the moisture in the RAP. The RAP is incorporated using three cold feed bins (Figure 2), while the RA was introduced through a separate pump system and blended in-line with the asphalt binder (Figure 3).

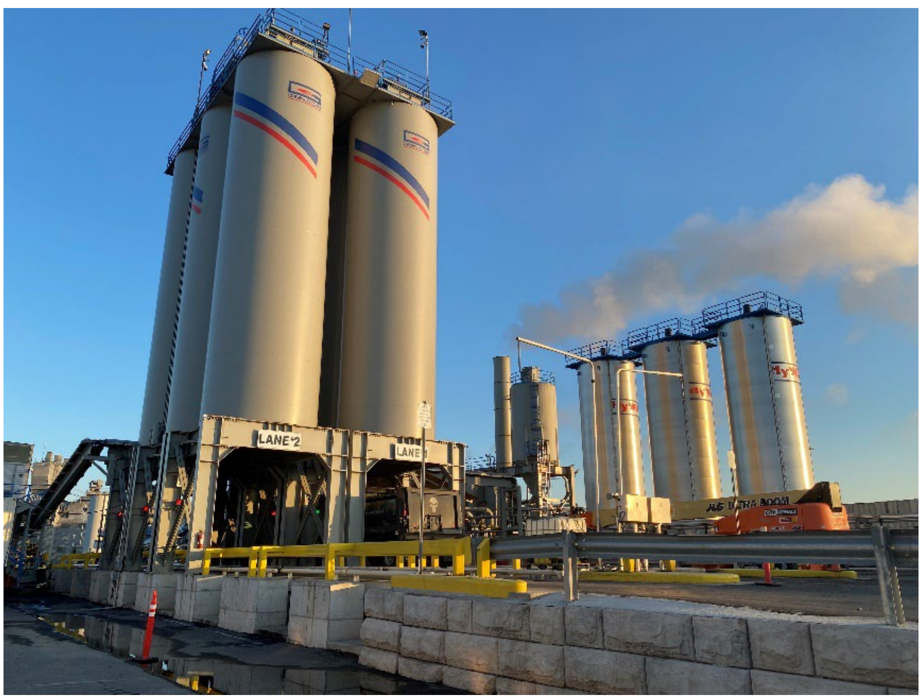


Figure 1 – City Asphalt Gencor Asphalt Plant

The dosage rates for the RA depended on the amount of RAP used in the asphalt mixture. The 45% RAP asphalt mix had an RA application of 1.8 lbs per ton while the 50% RAP asphalt mix used 2.0 lbs per ton.

Figure 2 – RAP Cold Feed Bins at City Asphalt

(a) **(b)**

Figure 3 – Liquid Recycling Agent Meter System; a) Totes of Liquid RA with Feed Lines; b) Pump and Metering System for Liquid RA

Prior to project, fine and coarse RAP, as well as the base PG64S-22 asphalt binder were provided to Rutgers University for characterization. Table 1 provides the asphalt binder properties of the PG64S-22 and the recovered asphalt binder from the coarse and fine RAP fraction.

Table 1 – Asphalt Binder Properties of RAP and Virgin Asphalt Binder

Binder Sample	Asphalt Content (%)	High Temp PG Grade		Intermediate Temp - 20 Hr PAV		Low Temp Grade - 20 Hr PAV		
		Original DSR	RTFO DSR	PG Grade	NCHRP 9-59 Glover-Rowe (kPa)	Tcr, S	Tcr, m	ΔTc
Base PG64S-22	N.A.	65.8	66.3	24.4	4510	-26.9	-24.7	-2.2
Fine RAP	5.74	91.1	89.4	34.1	20521	-20.6	-15.3	-5.3
Coarse RAP	3.29	88.9	87	31.4	16324	-22.8	-18.1	-4.7

In addition to the asphalt binder properties, the aggregate gradations and bulk specific gravity of the coarse and fine fraction were also characterized and provided to City Asphalt for use in the mixture design and production. Table 2 shows the aggregate properties of the RAP materials.

Table 2 – Aggregate Properties of City Asphalt RAP Materials

Sieve Size	% Finer	
	Coarse	Fine
3/4"	100.0	100.0
1/2"	97.5	100.0
3/8"	74.4	99.5
# 4	26.7	77.0
# 8	20.6	57.9
# 16	17.5	45.9
# 30	14.4	36.1
# 50	10.0	25.3
# 100	6.5	16.7
# 200	4.4	11.5
G_{sb} (g/cm^3)	2.754	2.738
G_{ssd} (g/cm^3)	2.787	2.763
G_{app} (g/cm^3)	2.848	2.808
ABS (%)	1.19	0.91
Asphalt Content (%) - T308	4.00	6.86
Asphalt Content (%) - T164	3.29	5.74
Ignition Oven Correction Factor	0.71	1.12

Plant production occurred between September 15th and September 28th, 2023. Table 3 shows the production dates, tonnage, and quality control data from each day's production. The 45% RAP Control and 50% RAP Control asphalt mixtures were not originally a part of this study. However, City Asphalt collected the loose mix and so mixture characterization was added at no cost. Overall, there was fairly good consistency between the different asphalt mixtures, especially when comparing the RA at each of the respective RAP contents. It should be noted that the plant air voids at the 50% RAP mixtures were lower than the 40% RAP Control and 45% RAP mixtures.

Table 3 – Plant Quality Control Test Results from NYCDOT High RAP Study

Date:	9/19/2023		9/20/2023		9/22/2023	9/27/2023		9/28/2023		JMF	
Tonnage:	601.7		604.7		583.4	608.4		1014.2			
Sample:	1	2	1	2	1	1	2	1	2		
Time:	8:17 AM	11:18 AM	8:19 AM	11:03 AM	7:42 AM	8:00 AM	10:46 AM	7:48 AM	9:44 AM	Targets	General Limits
Temp:	325 F	325 F	325 F	330 F	320 F	330 F	325 F	330 F	330 F		
3/4"	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0		100
1/2"	99.8	99.4	99.9	99.7	99.0	99.7	99.4	99.4	99.7		95-100
1/4	72.0	69.7	72.1	71.9	70.7	74.3	73.4	70.4	71.0		57-75
1/8	40.0	37.9	42.5	42.7	39.2	40.5	41.4	40.7	41.4		32-52
#20	21.8	21.1	24.2	24.4	20.8	21.8	22.3	23.3	23.4		15-32
#40	16.4	15.9	18.2	18.4	14.9	16.4	16.7	17.7	17.5		8-25
#80	10.2	10.1	11.1	11.1	8.0	10.5	10.4	11.1	10.8		5-17
#200	6.6	6.7	7.0	7.0	4.3	6.9	6.6	7.2	6.8		2-8
AC%	5.25	5.20	5.21	5.35	5.28	5.30	5.30	5.33	N/A	5.3	5.0-6.2
Rice (pcf)	161.7	162.1	161.4	160.7	161.8	161.7	162.0	161.1	160.9	N/A	N/A
Rice (g/cm3)	2.591	2.598	2.587	2.575	2.593	2.591	2.596	2.582	2.579		
Rice (g/cm3)	2.595		2.581			2.594		2.580			
Air Voids	3.3	3.6	1.8	1.5	3.1	2.9	2.8	1.7	1.7	4.0	3.0 - 5.0
Ave. Gmb	2.505		2.538		2.513	2.520		2.536			
VMA (%)	13.7		12.6		13.5	13.2		12.7			
Eff AC%	10.3		10.9		10.4	10.3		11.0			
Stability	2690	2710	3480	3470	3140	3200	3380	3470	3600		> 1800
Flow	10.7	10.3	11.1	11.3	11.0	10.6	10.3	10.9	11.3		8-16
Mix Type	**45% RAP + Soylei**		**50% RAP + Soylei**		**40% RAP + No RA**	**45% RAP + No RA**		**50% RAP + No RA**		JMF	

RESEARCH APPROACH

Loose mix from each day's production was sampled from the bed of the delivery truck at the sampling stand prior to leaving the asphalt plant (Figure 4). Sampling occurred between approximately 125 to 200 tons of produced loose mix to ensure the plant had a chance to stabilize and produce a consistent mix. Approximately 10 to 12 5-gallon buckets of loose mix for each mix were sampled for laboratory characterization.

Figure 4 – Sampling Loose Mix from Delivery Truck at Sampling Stand

The laboratory characterization occurred in two phases; 1) Asphalt mixture performance testing on compacted asphalt specimens and 2) Asphalt binder characterization of recovered asphalt binder. Both the recovered asphalt binder and loose mix underwent extended laboratory aging to evaluate the cracking/durability properties of the asphalt materials.

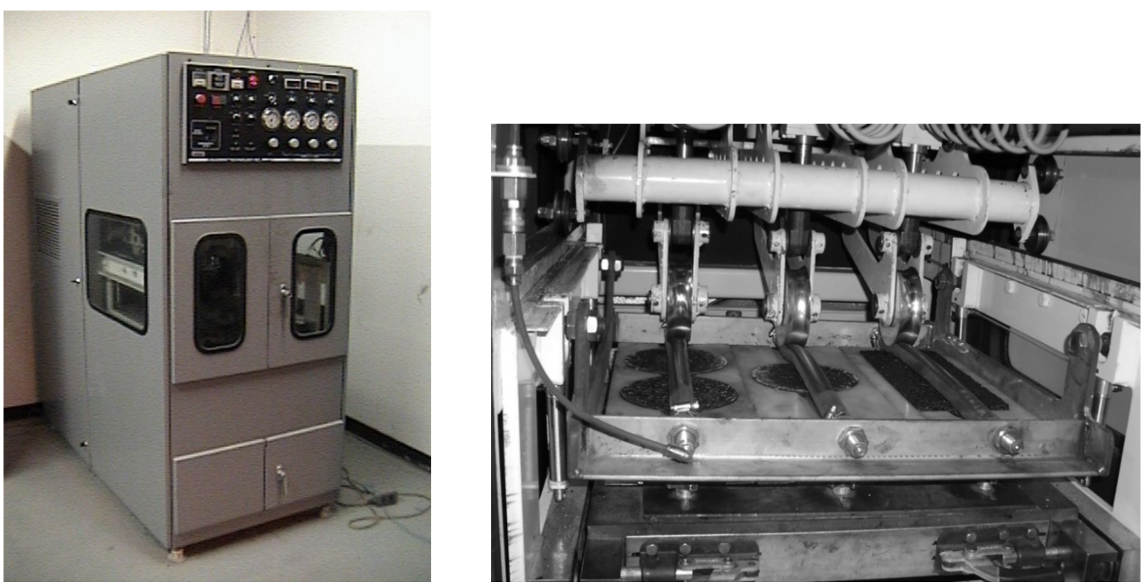
Research Workplan – Asphalt Mixture Testing

Asphalt mixture characterization consisted of evaluating the mixture stiffness, rutting, and fatigue cracking performance. The asphalt mixtures were laboratory conditioned to provide two different levels of asphalt mixture aging. All asphalt mixtures were conditioned under "Short-term oven aging, STOA", which consisted of reheating the loose mix for 2 hours at compaction temperature prior to specimen compaction. In addition, test specimens evaluated for stiffness and fatigue cracking were also laboratory conditioned for "Long-term oven aging, LTOA" after the STOA had completed. LTOA consisted of loose mix conditioning at 95ºC for 4 days in a shallow pan, in accordance with the recommendation of NCHRP Project 9-54 (Kim, et al., 2018). Based on the recommendations of NCHRP 9-54, the four days of loose mix conditioning is equivalent to approximately 4 to 5 years of field aging at a depth of 6 mm or approximately 8 to 9 years at a depth of 20 mm below the surface in the New York City area. Personal experience at Rutgers University has shown that when laboratory conditioning beyond the 4 days at 95ºC, differentiating between the fatigue cracking performance of LTOA asphalt mixtures is difficult and often leads to mixtures that behave identically. Therefore, the 4 days at 95ºC was selected to assess the impact of the RAP and RA on the "aged" asphalt mixture stiffness and cracking potential. All asphalt mixture test specimens were compacted to a target air void level of 6.5%, +/- 0.5% air voids.

Asphalt Pavement Analyzer (AASHTO T340)

The Asphalt Pavement Analyzer (APA) was conducted in accordance with AASHTO T340, *Determining Rutting Susceptibility of Asphalt Paving Mixtures Using the Asphalt Pavement Analyzer (APA).* A hose pressure of 100 psi and a wheel load of 100 lb was used in the testing. Testing was continued until 8,000 loading cycles and APA rutting deformation was recorded at each cycle. The APA device used for testing at Rutgers University is shown in Figure 5.

Prior to testing, the specimens were heated for 6 hours (+/- 15 minutes) at the testing temperature to ensure temperature equilibrium within the test specimen was achieved. Testing started with 25 cycles used as a seating load to eliminate any sample movement during testing. After the 25 seating cycles completed, the data acquisition began sampling test information until a final 8,000 loading cycles was reached.



(a) (b)

Figure 5 - a) Asphalt Pavement Analyzer (APA) at Rutgers University; b) Inside the Asphalt Pavement Analyzer Device

High Temperature Indirect Tension Test (HT-IDT)

The IDT test was originally developed by Carneiro (1943) when he proposed that the test method could be used to determine the tensile strength of concrete. Similar research was also being conducted in Japan by Akazawa (1943), but due to World War II, published information related to the work in Japan became unvailable. Carneiro (1943) determined that by testing the concrete specimen in a diametral position, a relatively uniform tensile stress develops perpendicular to and along the diametral. Prior to 1962, the IDT test was primarily used for concrete. It was not until after 1962 that other researchers began utilizing the IDT test method for other materials. Livneh and Shklarsky (1962) reported the use of the IDT test to evaluate the anisotropic cohesion of asphalt mixtures. Thompson (1965) began utilizing the test method to evaluate lime stabilized soils and asphalt mixtures. Soon after in 1966, Messina (1966) conducted research at the University of Texas utilizing the IDT procedure to again evaluate asphalt mixtures. In the same year, Breen and Stephens (1966) also began utilizing the IDT procedure to evaluate the low temperature tensile properties of asphalt mixtures. A series of reports conducted by Kennedy and his associates at the University of Texas critically evaluated the IDT test procedure for asphalt materials under both static and dynamic loading conditions and made the IDT an accepted test method to evaluate asphalt mixtures (Kennedy and Anagnos, 1966).

More recently, under the movement to develop a "simple" performance test for asphalt mixtures, researchers again re-evaluated the IDT test and its relationship to various asphalt mixture properties. Gokhale (2001) evaluated the use of the IDT test method to better understand the shear strength properties of asphalt mixtures and concluded that the IDT strength was highly correlated to asphalt mixture cohesion under the Mohr-Coulomb shear strength failure envelope theory. Christensen et al., (2004) found that the IDT test, conducted at a test temperature representative of the critical pavement temperature, was found to be a simple, inexpensive, and effective test for evaluating the rutting resistance of asphalt mixtures. The authors also provided preliminary threshold values that were developed based on correlations from testing FHWA ALF mixtures. Unfortunately, the final test method used a loading rate of 3.75 mm/min, requiring a more specialized loading system. However, utilizing updated models and test data from NCHRP projects 9-25, 9-31, and 9-33, Christensen and Bonaquist refined the test method to be conducted at 50 mm/min and a test temperature 10ºC lower than the yearly, 7-day average, maximum pavement temperature 20 mm below the pavement surface as determined by LTPPBind (Christensen and Bonaquist, 2007; Advanced Asphalt Technologies (AAT), 2011). These changes greatly improved the IDT test, now called High Temperature IDT (HT-IDT), by providing standardized guidance on test temperature, as well as allowing more readily available test equipment to conduct the test. For this study, a test temperature of 44ºC was used during the HT-IDT testing.

Overlay Tester (NJDOT B-10)

The Overlay Tester has shown to provide an excellent correlation to field cracking for both composite pavements as well as flexible pavements. Figure 6 shows a picture of the Overlay Tester used in this study. Sample preparation and test parameters used in this study followed that of NJDOT B-10, *Overlay Test for Determining Crack Resistance of HMA*. These included:

- 25ºC (77ºF) test temperature;
- Opening width of 0.025 inches;
- Cycle time of 10 seconds (5 seconds loading, 5 seconds unloading); and
- Specimen failure defined as 93% reduction in Initial Load.

Test specimens were evaluated under both short-term and long-term aged conditions.

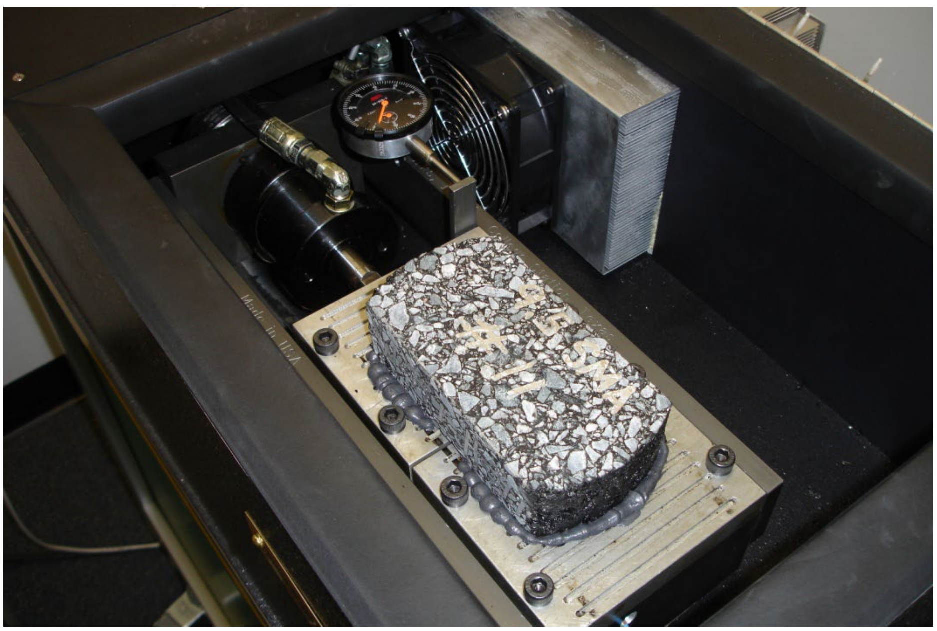
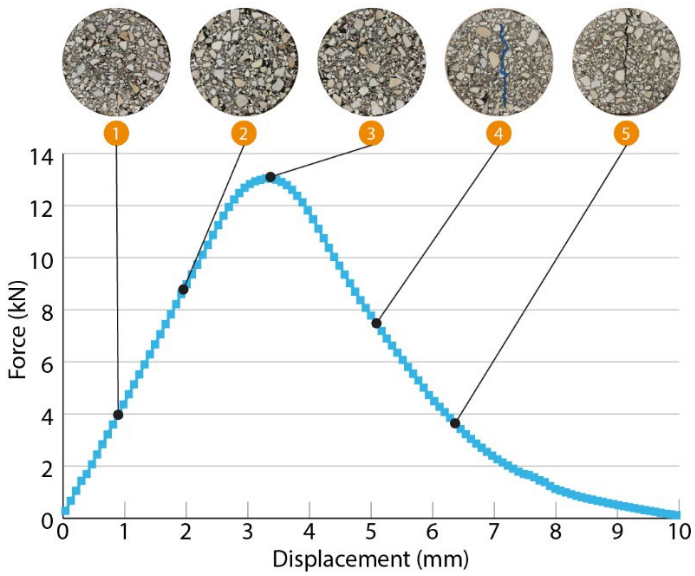


Figure 6 – Picture of the Overlay Tester (Chamber Door Open)

IDEAL-CT Fatigue Cracking Test

The IDEAL-CT is similar to the traditional indirect tensile strength test, and it is run at the room temperature with cylindrical specimens at a loading rate of 50 mm/min. in terms of cross-head displacement. Any size of cylindrical specimens with various diameters (100 or 150 mm) and thicknesses (38, 50, 62, 75 mm, etc.) can be tested. For mix design and laboratory QC/QA, the authors proposed to use the same specimen size as the Hamburg wheel tracking test: 150 mm diameter and 62 mm height, since agencies are familiar with molding such specimens. Either lab-molded cylindrical specimens or field cores can be directly tested with no need for instrumentation, gluing, cutting, notching, coring or any other preparation.

Figure 7 shows a typical IDEAL-CT: cylindrical specimen, test fixture, test temperature, loading rate, and the measured load vs. displacement curve.



Test temperature: 25°C

Loading rate: 50 mm/min.

Specimen: cylindrical specimen without cutting, gluing, instrumentation, drilling, or notching.

Figure 7 - IDEAL-CT: Specimen, Fixture, Test Conditions, and Typical Result

After carefully examining the typical load-displacement curve and associated specimen conditions at different stages (Figure 7), the authors chose the post-peak segment to extract cracking resistance property of asphalt mixes. Note that with the initiation and growth of the macro-crack, load bearing capacity of any asphalt mix will obviously decrease, which is the characteristic of the post-peak segment. The calculation for the cracking parameter, named CT_{Index}, is shown in Equation 1.

$$CT_{Index} = \frac{G_f}{|m_{75}|} \times \left(\frac{l_{75}}{D}\right) \tag{1}$$

where Gf is the energy required to create a unit surface area of a crack; $|m_{75}| = \left|\frac{P_{85}-P_{65}}{l_{85}-l_{65}}\right|$ is the secant slope is defined between the 85 and 65 percent of the peak load point of the load-displacement curve after the peak; and l_{75} is deformation tolerance at 75 percent maximum load.

Generally, the larger the Gf, the better the cracking resistance of asphalt mixes. The stiffer the mix, the faster the cracking growth, the faster the load reduction, the higher the $|m_{75}|$ value, and consequently the poorer the cracking resistance. It is obvious that the mix with a larger $\frac{l_{75}}{D}$ and better *strain* tolerance has a higher cracking resistance than the mix with a smaller $\frac{l_{75}}{D}$.

Dynamic Modulus (Stiffness)

The stiffness characteristics of the asphalt mixtures were evaluated by measuring the dynamic modulus and phase angle properties in uniaxial compression using the Asphalt Mixture Performance Tester (AMPT) following the method outlined in AASHTO T378*, Determining the Dynamic Modulus and Flow Number for Hot Mix Asphalt (HMA) Using the Asphalt Mixture Performance Tester (AMPT)*. The data was collected at three temperatures; 4, 20, 35 and 45°C using loading frequencies of 25, 10, 5, 1, 0.5, 0.1, and 0.01 Hz. Test specimens were evaluated under both STOA and LTOA conditions. The collected modulus values of the varying temperatures and loading frequencies were used to develop dynamic modulus master stiffness curves and temperature shift factors using numerical optimization of Equations 2 and 3. The reference temperature used for the generation of the master curves and the shift factors was 20°C.

$$\log|E^*| = \delta + \frac{(Max - \delta)}{1 + e^{\beta + \gamma\left\{\log \omega + \frac{\Delta E_a}{19.14714}\left[\left(\frac{1}{T}\right) - \left(\frac{1}{T_r}\right)\right]\right\}}} \qquad (2)$$

where:

$|E^*|$ = dynamic modulus, psi
ω_r = reduced frequency, Hz
Max = limiting maximum modulus, psi
δ, β, and γ = fitting parameters

$$\log[a(T)] = \frac{\Delta E_a}{19.14714}\left(\frac{1}{T} - \frac{1}{T_r}\right) \qquad (3)$$

where:

a(T) = shift factor at temperature T
T_r = reference temperature, °K
T = test temperature, °K
ΔE_a = activation energy (treated as a fitting parameter)

Research Workplan – Asphalt Binder Testing

The asphalt binder was extracted and recovered in accordance with AASHTO T319, *Standard Method of Test for Quantitative Extraction and Recovery of Asphalt Binder from Asphalt Mixtures* using tri-chloroethylene (TCE) as the solvent medium (Figure 8). After recovery, the asphalt binder was tested for its respective PG grade, in accordance with AASHTO M320, *Standard Specification for Performance-Graded Asphalt Binder* and Multiple Stress Creep Recovery (MSCR) in accordance with AASHTO T350, *Standard Test Method for Multiple Stress Creep Recovery (MSCR) Test of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)*. Low temperature performance grading was also conducted after an additional 20 hour cycle in the PAV (40 hours total) to assess the impact of extended aging on the low temperature PG grade and ΔTc parameter.

Figure 8 – Asphalt Binder Recovery Equipment at Rutgers University

In addition to the conventional asphalt binder grading, the following asphalt binder tests parameters/tests were also conducted.

<u>Glover-Rowe Parameter (G-R)</u>

Glover et al. (2005) proposed the rheological parameter, G'/(η'/ G'), as an indicator of ductility based on a derivation of a mechanical analog consisting of springs and dashpots to represent the traditional ductility test. It has been well demonstrated that the Glover parameter is directly correlated to measured ductility. The Glover parameter can be calculated based on DSR frequency sweep testing results, making it much more practical than directly measuring ductility using traditional methods. Rowe (2011) re-defined the Glover parameter in terms of |G*| and δ based on analysis of a black space diagram as shown in Equation (4) and suggested use of the parameter $|G^*|\cdot(\cos\delta)^2/\sin\delta$, termed the Glover-Rowe (G-R) parameter in place of the original Glover parameter and measured at 15ºC and 0.005 radians/sec.

$$\frac{G'}{\eta'/G'} = \frac{|G^*|\cdot(\cos\delta)^2}{\sin\delta}\cdot\omega \tag{4}$$

A higher G-R value indicates increased brittleness. It has been proposed that a G-R parameter value of 180 kPa corresponds to damage onset whereas a G-R value exceeding 600 kPa corresponds to significant cracking based on a study relating binder ductility to field block cracking and surface raveling by Anderson et al. (2015).

The Glover-Rowe parameter was also measured at a test temperature of 25ºC and at 10 radians/sec. This is based on the recommendations of Christensen and Tran (2018) and provides a more consistent approach to what is currently being done for intermediate PG grading under AASHTO M320. Preliminary recommendations note that asphalt binders should not have a value greater than 5,000 kPa after 20 hour PAV conditioning nor greater than 8,000 kPa after 40 hour of PAV conditioning.

<u>Asphalt Binder Cracking Device (ABCD) – AASHTO T387</u>

The Asphalt Binder Cracking Device (ABCD) was utilized to evaluate the low temperature critical cracking temperature (T_{cr}) of the asphalt binders due to thermally induced (cooling) stress. The ABCD T_{cr} is primarily controlled by the coefficient of thermal contraction (CTC) of the asphalt binder. The CTC controls the rate of volumetric change in the asphalt binder, thereby controlling the rate of thermal stress development. An asphalt binder with a higher CTC may be subjected to larger strains compared to low CTC asphalt binders before cracking failure is observed. Work conducted under NCHRP Project 9-60 showed that the asphalt binder's CTC affects non-load related cracking (Elwardany et al., (2020)). To better interpret the ABCD data, Elwardany et al., (2020) recommended normalizing the ABCD T_{cr} using the stiffness-based low temperature grade from the bending beam rheometer (BBR) test (Tc = 300 MPa). This was recommended for two reasons;

1. The T_C (S = 300 MPa) is highly correlated with the glass transition temperature, T_g. Modifiers like REOB will actually help to reduce the T_g. Therefore, normalizing the ABCD T_{cr} using T_C (S = 300 MPa) should improve the sensitivity of the ABCD cracking data to REOB-type modification that reduces T_g.
2. Since the T_C (S = 300 MPa) is already a part of the PG grading system, it is simpler to use than the actual T_g value, which would require sophisticated measurement equipment outside of conventional asphalt binder test equipment.

Therefore, Elwardany et al., (2020) proposed the parameter ΔT_f, described below.

$$\Delta T_f = T_C(S) - T_{cr} \quad (5)$$

where,

$T_C(S)$ = low temperature PG grade from BBR Stiffness

T_{cr} = ABCD low temperature critical cracking temperature

The ΔT_f parameter is combined with the ΔT_c parameter from the low temperature BBR testing to provide a performance space for evaluating the potential for non-load associated cracking (Figure 9).

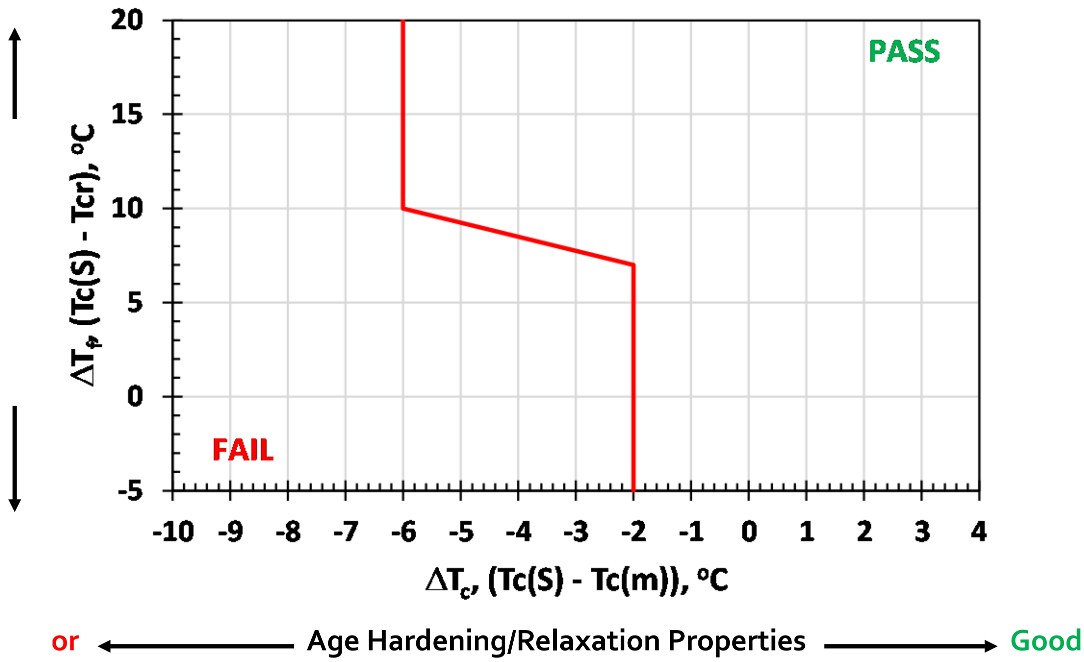


Figure 9 – NCHRP 9-60 Performance Space Using BBR ΔT_c and ABCD ΔT_f for 20 Hour PAV Conditioned Asphalt Binders

ASPHALT MIXTURE TEST RESULTS

The asphalt mixture test specimens were produced to a target air voids of 6.5% +/- 0.5% to mirror typical in-field density levels. All specimens utilized for rutting evaluation were STOA, while stiffness and fatigue cracking were STOA + LTOA conditioned using the methodologies described earlier.

Rutting Resistance

The results of the Asphalt Pavement Analyzer (AASHTO T340) rutting are shown in Figure 10. In general, the results show the 40% RAP Control mixture having the lowest rutting potential, while the other mixtures show similar results. Evaluating the test data using the statistical t-Test at a 95% confidence interval, the 45% and 50% RAP mixtures were found to be statistically equal.

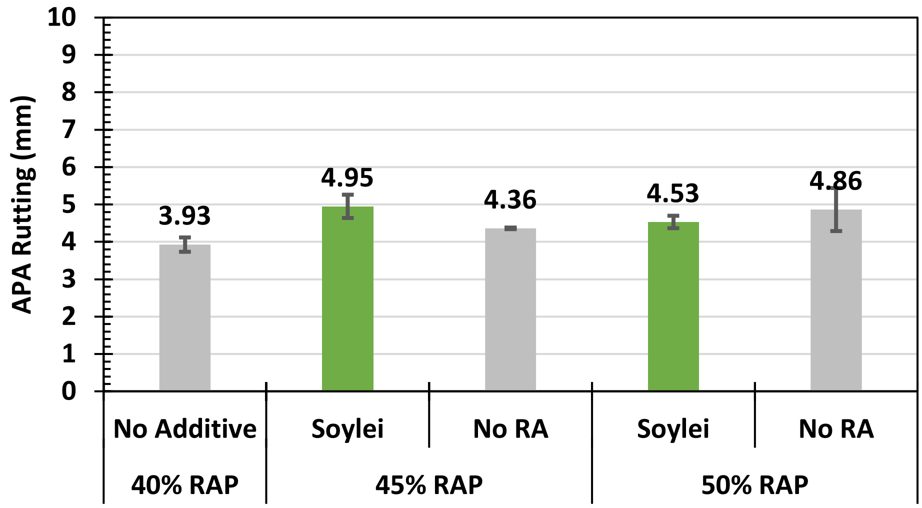


Figure 10 – Asphalt Pavement Analyzer (APA) Rutting Results

The high temperature Indirect Tensile Strength (HT-IDT) test results can be found in Figure 11. Similar to the APA rutting, the 40% RAP Control mixture showed the highest HT-IDT strength, and therefore, the most rut resistant. Again, using the t-Test to compare the different RAP contents, the 45% RAP with Soylei and 50% RAP with Soylei were found to have a statistically equal HT-IDT strength, with both of the RA mixtures having a statistically lower HT-IDT strength to the identical mixtures without RA.

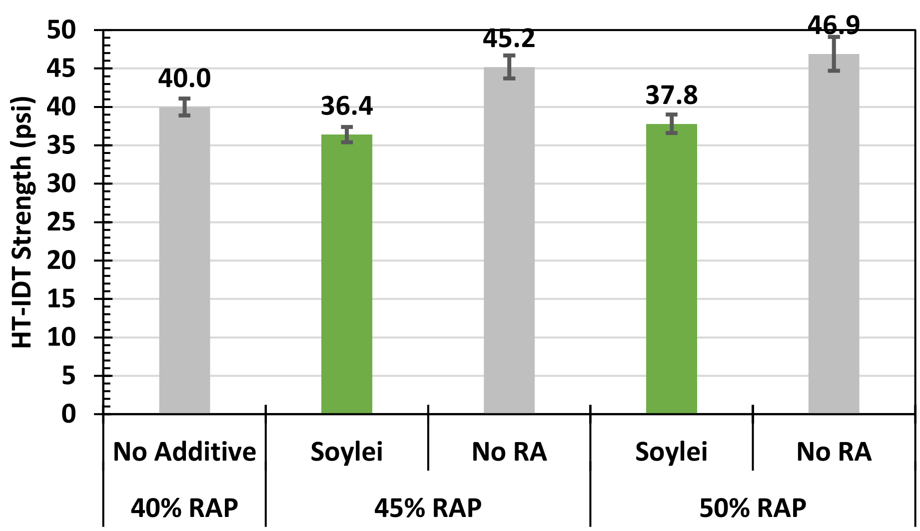


Figure 11 – High Temperature Indirect Tensile Strength (HT-IDT) Test Results

Fatigue Cracking Resistance

The results of the IDEAL-CT Index tests are shown in Figure 12. For the STOA conditioned mixes, on average, the Soylei RA at 45% RAP achieved the highest IDEAL-CT Index value with all other STOA mixtures resulting in statistical equal IDEAL-CT Index performance. For the LTOA IDEAL-CT Index specimens, although on average the 45% RAP Soylei RA mixture achieved the highest IDEAL-CT Index, the associated variability among all mixes resulted in all 5 mixes being statistically equal at a 95% confidence interval using the t-Test.

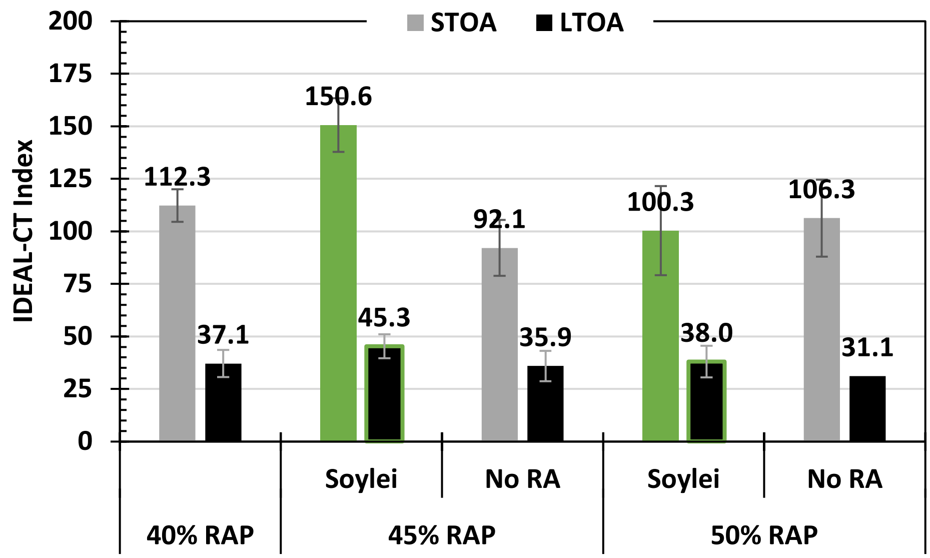


Figure 12 – IDEAL-CT Index Test Results

The Overlay Tester results are shown in Figures 13 and 14. Figure 13 analyzes the test data using the Number of Cycles to Failure approach, while Figure 14 using the Crack Propagation Rate approach. For the Number of Cycles to Failure, on average the 40% RAP Control mixture resulted in the best Cycles to Failure. However, the t-Test statistical analysis showed that all 5 mixes, when using the Number of Cycles to Failure approach, were statistically equal at a 95% confidence interval. Meanwhile, when using the Overlay Tester Crack Propagation Rate approach, the 40% RAP Control and both 45% RAP Soylei and 50% Soylei mixes were found to be statistically equal and have better cracking performance than the 45% RAP and 50% RAP mixes without RA.

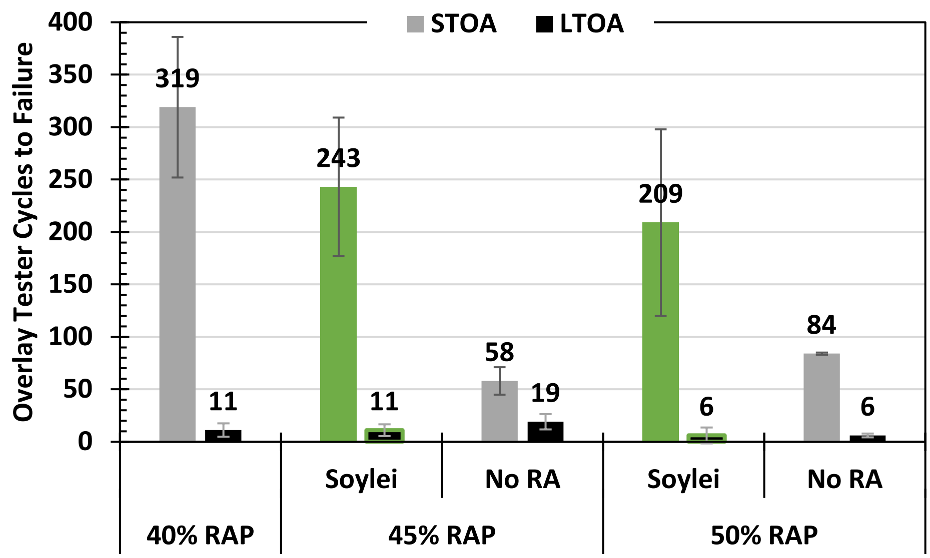


Figure 13 – Overlay Tester Cycles to Failure Test Results

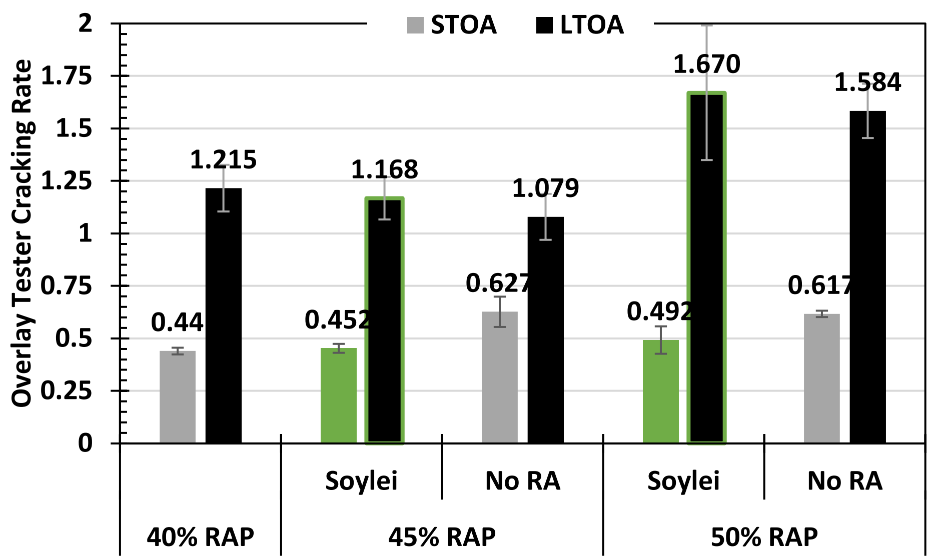


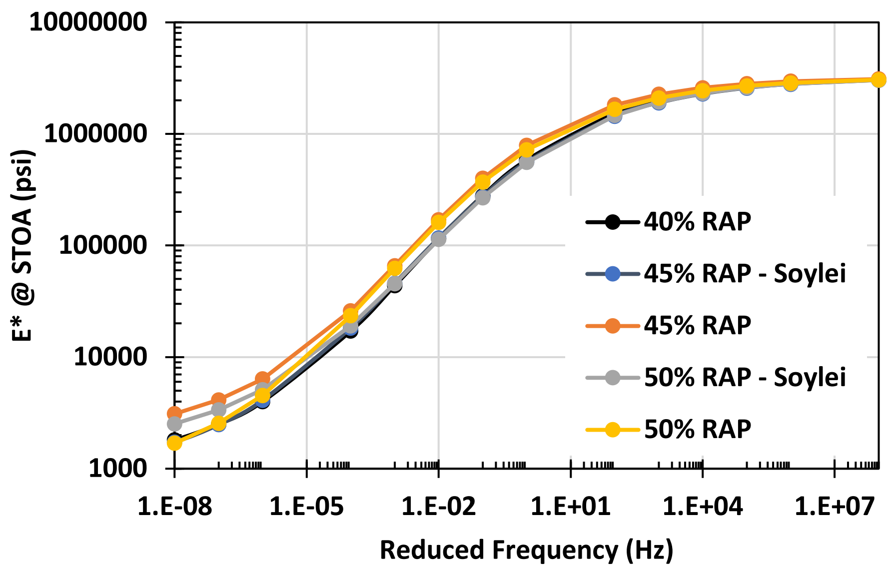
Figure 14 – Overlay Tester Crack Propagation Rate Test Results

Asphalt Mixture Stiffness

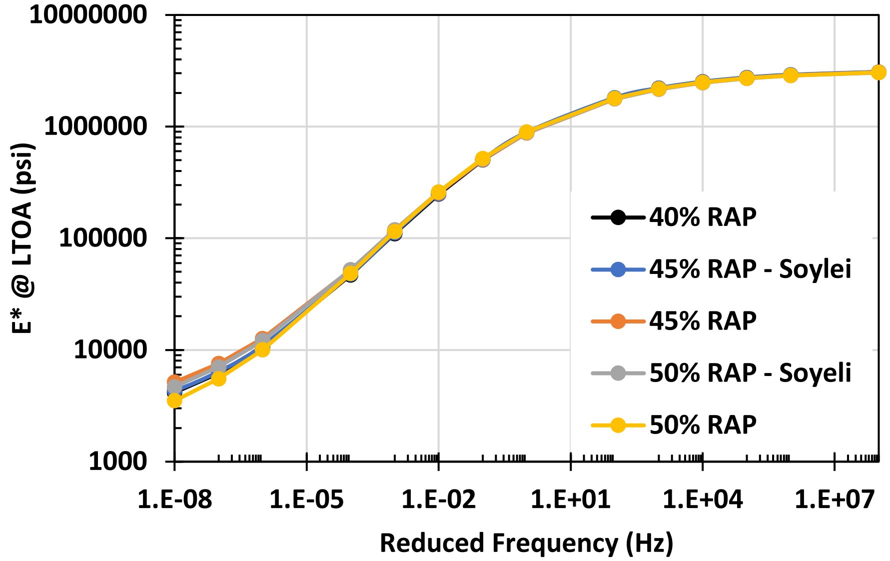
The stiffness of the high RAP asphalt mixtures was evaluated using the Dynamic Modulus test (AASHTO T387). The asphalt mixtures were evaluated under a range of temperatures and loading rates to help develop a "master stiffness curve" which is essentially a fingerprint of the asphalt material describing its stiffness behavior over a wide range of temperature and loading rates that can be implemented in asphalt material and pavement design/modeling.

The master stiffness curves for the STOA conditioned mixtures are shown in Figures 15a and 15b. As a frame of reference, data points to the left of the curve represents warm temperatures and slow loading rates. Meanwhile, data points to the right of the curves represent low temperatures and fast loading rates. And as can be concluded, the middle part of the curves represents intermediate temperatures and moderate loading rates. Higher stiffness at high temperatures is beneficial to reduce rutting potential while lower stiffness at lower and intermediate temperatures is beneficial to reduce low and intermediate temperature cracking.

Figure 15a and 15b shows the master stiffness curves for the different asphalt mixtures after STOA and LTOA conditioning, respectively. The results show that low and intermediate test temperature ranges are equivalent among the five asphalt mixtures. It is not until the higher test temperatures where the 45% RAP without RA mixture and 50% RAP with Soylei appear to have highest stiffness. The remaining mixtures all appear to have similar dynamic modulus results.



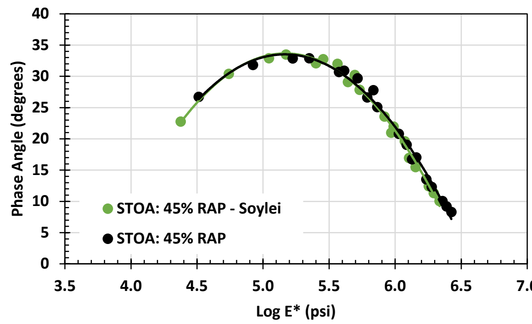
(a)



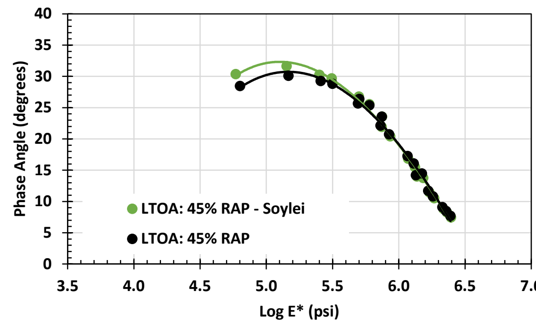
(b)

Figure 15 – Dynamic Modulus (E*) Master Stiffness Curves; a) STOA Conditioned and b) LTOA Conditioned

Figure 16 shows the dynamic modulus and phase angle measurements of the 45% RAP mixtures within a Black Space diagram (BSD). The BSD is a method to represent rheological data to help identify potential non-linearity in test data, as well as identify the impact of aging, additives etc. on the stiffness properties of the asphalt material. Within the BSD as shown in Figure 16, as the test data moves towards the upper left part of the diagram (i.e. – higher phase angle, lower dynamic modulus), the asphalt materials become softer. As opposed to moving towards the lower right (i.e. – lower phase angle, higher dynamic modulus) where more age hardening/stiffening occurs. For the 45% RAP mixes, there is a slight progression towards softening of the asphalt mixture for the Soylei modified mixture, especially at the long term aged condition (LTOA).



(a)



(b)

Figure 16 – Black Space Diagram Using Asphalt Mixture Dynamic Modulus (E*) and Phase Angle (ϕ) for 45% RAP Mixes; a) STOA Conditioned; b) LTOA Conditioned

Meanwhile, Figure 17 shows the 50% RAP mixtures. In this case, the rheological response is similar with even the 50% RAP mixture showing it is slightly softer at the short term aged condition (STOA) but identical at the LTOA.

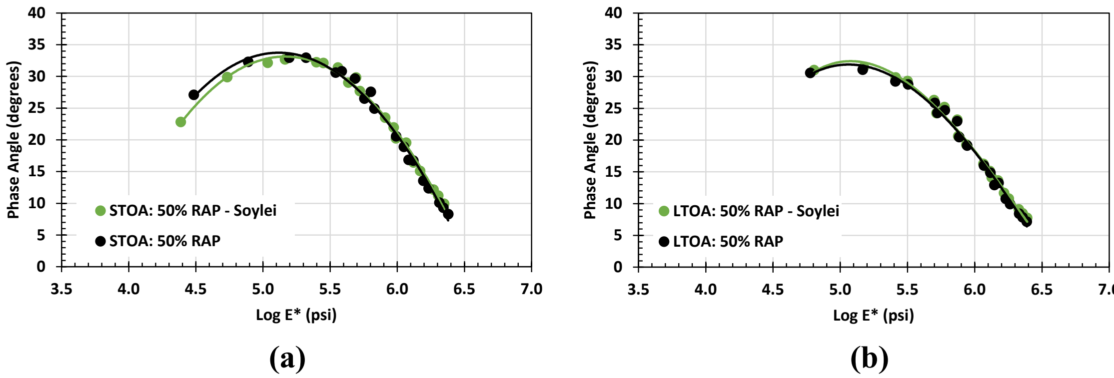


Figure 17 - Black Space Diagram Using Asphalt Mixture Dynamic Modulus (E*) and Phase Angle (ϕ) for 50% RAP Mixes; a) STOA Conditioned; b) LTOA Conditioned

BALANCED MIXTURE DESIGN (BMD) EVALUATION

Due to their reliance on volumetric measurements, current mixture design practices are not capable of addressing the impact recycled asphalt, recycling agents, modifiers, etc. To help address the lack of sensitivity observed in volumetric design, Balanced Mixture Design (BMD) has grown in popularity over the past decade. BMD incorporates rutting and cracking tests within the mixture design process, either as a Pass/Fail criteria to validate volumetric practices or in the final selection of the asphalt content to be utilized during production.

The New York State Department of Transportation (NYSDOT) is moving towards incorporating performance test requirements during mixture design and plant production in an effort to implement BMD within New York. At the time of this report, test procedures and performance criteria have not yet been finalized in New York, however, NYSDOT is evaluating the following test procedures and associated criteria (Bittner et al., 2023);

- Rutting
  - High Temperature Indirect Tensile (HT-IDT) Strength: > 30 psi at 44°C
- Fatigue Cracking
  - IDEAL-CT Cracking Index: > 135 at 25°C
  - SCB Flexibility Index: > 8.0 at 25°C

Using the HT-IDT and IDEAL-CT Index collected in this study, the BMD criteria utilized by NYSDOT was used to develop a Performance Space for the high RAP mixtures tested in this study (Figure 18). The results shown in Figure 18 note the 45% RAP mix with Soylei would have met both rutting and cracking performance while the other mixtures would have not met the minimum fatigue cracking requirements of NYSDOT's BMD approach.

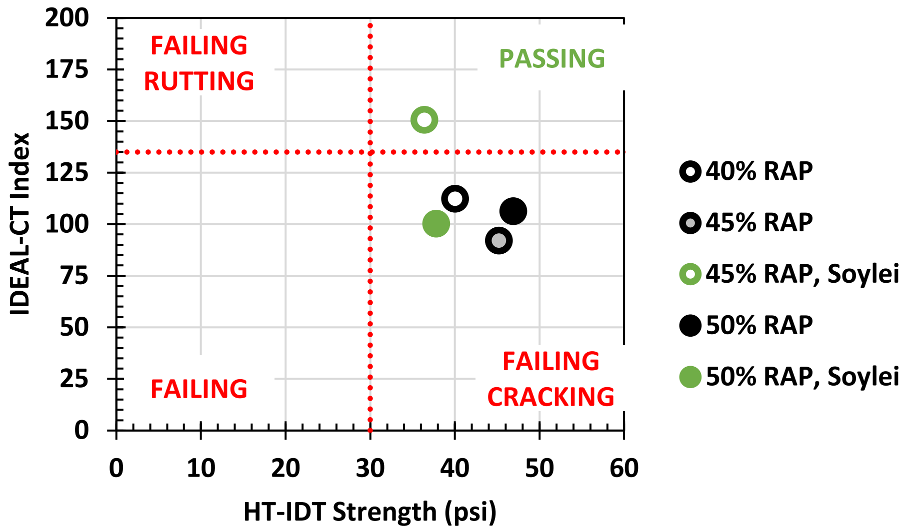


Figure 18 – Rutting and Fatigue Cracking Performance Space Using NYSDOT Criteria

It should be noted that currently, the NYCDOT requires a minimum of 40% RAP to be used in the asphalt mixtures placed in New York City. However, based on the performance testing and NYSDOT's BMD criteria, the 40% RAP would not be able to be used. Although, the test results clearly indicate that up to 45% RAP with the Soylei recycling agent would meet the minimum requirement set by the New York State Department of Transportation.

ASPHALT BINDER TEST RESULTS

Asphalt binder was recovered from the collected loose mix and evaluated for the respective performance grade properties, as well as their rheological and fracture toughness. It should be noted that the recovered asphalt binder content for the 40% RAP mixture was slightly higher than the RA asphalt mixtures by approximately 0.3% on average using solvent extraction and recovery procedures mentioned earlier in the report. In addition, the asphalt binder properties of the 45% RAP without RA and 50% RAP without RA mixtures were not measured as it was not part of the overall scope of the study. The asphalt mixture data shown earlier was conducted "gratis" as the material was produced and sampled by City Asphalt at the request of NYCDOT. Therefore, only the asphalt binder properties of the 40% RAP, 45% RAP with Soylei and 50% RAP with Soylei are shown.

Performance Grade Results

The Performance Grade results are shown in Tables 4 through 6. The high temperature PG grade results show all 3 recovered asphalt binders to be within 2°C of one another. The intermediate temperature grade was within 3.9°C of one another, with the 40% RAP Control mix resulting in the highest intermediate temperature grade. The low temperature PG grade were all within 1.7°C of one another and all met the -22°C low temperature PG grade requirement for the New York City climatic condition. Overall, there was rather good consistency in the PG grade properties of the recovered asphalt binders for all RAP contents evaluated.

Table 4 – High Temperature Performance Grade Results

Mix Type	Asphalt Content (AASHTO T319)	Dynamic Shear Rheometer (AASHTO T315/R29) & Continuous High Temp PG Grade (ASTM D7643)								Multiple Stress Creep Recovery (AASHTO T350)			
		Parameter	Test Results at T_1		Test Results at T_2		T_C (°C)	Final PG Grade (°C)	64 °C			
			T_1 (°C)	P_1	T_2 (°C)	P_2			Jnr (1/kPa)	% Rec	Elastomer	Grade
Control 40% RAP	5.31	G* (kPa)	70	4.157	76	1.998	75.3	70.0	0.816	7.7	FAIL	V
		δ (degrees)		80.78		83.40						
		G*/sin δ (kPa)		4.211		2.011						
Soylei 45% RAP	5.18	G* (kPa)	70	3.245	76	1.615	73.4	70.0	1.182	5.0	FAIL	H
		δ (degrees)		81.40		83.76						
		G*/sin δ (kPa)		3.282		1.625						
Soylei 50% RAP	4.97	G* (kPa)	70	3.972	76	1.964	75.1	70.0	1.000	5.9	FAIL	V
		δ (degrees)		80.01		82.63						
		G*/sin δ (kPa)		4.033		1.980						

Table 5 – Intermediate Temperature Performance Grade Results

Mixture Type	Aged Condition	Intermediate Temp PG Grade		
		Intermed. Temp PG Grade (^oC)	Glover-Rowe Parameter (kPa)	
			Traditional	NCHRP 9-59 @ 25C
Control 40% RAP	20 Hr PAV	27.1	336	8586
	40 Hr PAV	N.A.	1307	16190
45% RAP - Soylei	20 Hr PAV	23.2	81	3342
	40 Hr PAV	N.A.	754	10690
50% RAP - Soylei	20 Hr PAV	24.9	271	6940
	40 Hr PAV	N.A.	1315	14930

Table 6 – Low Temperature PG Grade and Cracking Test Results

Mixture Type	Aged Condition	BBR Parameter	Bending Beam Rheometer Data (AASHTO T313/R29) & Continuous Low Temp PG Grade (ASTM D7643)							Asphalt Binder Cracking Device	
			Test Results at T_1		Test Results at T_2		T_C (°C)	Final T_C (°C)	ΔT_C	T_{cr} (^oC)	ΔT_f
			T_1 (°C)	P_1	T_2 (°C)	P_2					
Control 40% RAP	20 Hr PAV	S (MPa)	-18	373	-12	209.5	**-25.7**	**-23.4**	**-2.3**	**-27.2**	**1.5**
		m-value	-18	0.264	-12	0.311	**-23.4**				
	40 Hr PAV	S (MPa)	-18	423	-12	245	**-24.2**	**-15.5**	**-8.7**	**-26.3**	**2.1**
		m-value	-6	0.297	0	0.336	**-15.5**				
Soylei 45% RAP	20 Hr PAV	S (MPa)	-18	353	-12	175.5	**-26.6**	**-25.1**	**-1.5**	**-30.4**	**3.8**
		m-value	-18	0.276	-12	0.326	**-25.1**				
	40 Hr PAV	S (MPa)	-18	387.5	-12	233	**-25.0**	**-15.6**	**-9.4**	**-26.4**	**1.4**
		m-value	-6	0.297	0	0.349	**-15.6**				
Soylei 50% RAP	20 Hr PAV	S (MPa)	-18	344	-12	166	**-26.9**	**-24.2**	**-2.7**	**-28.7**	**1.8**
		m-value	-18	0.267	-12	0.320	**-24.2**				
	40 Hr PAV	S (MPa)	-18	402	-12	0.267	**-27.8**	**-16.9**	**-10.9**	**-25.1**	**-2.7**
		m-value	-12	0.267	0	0.344	**-16.9**				

Rheological Properties

To evaluate the Glover Rowe rheological indices in the study, each asphalt binder was tested for their respective master curve stiffness properties. Both G* and phase angle (δ) were determined at a various test temperature and loading frequencies. As there are currently no standardized procedures in either AASHTO or ASTM for determining the master stiffness properties of asphalt binders, the general procedure used follows that recommended by Rowe (2015) and summarized below;

Using the DSR in the oscillatory mode and within the strain range 0.005 to 0.02 (± 5%), the test specimens were tested over the linear region over the temperature range chosen. A linearity check was carried out by a torque sweep at both the highest and lowest test temperature to be used for the rheological characterization. For most binders, it has been found that testing within the strain range 0.005 to 0.02 lies within the linear range, although polymer modified binders may be less. The linear range available depends upon the stiffness of a binder at the condition being evaluated. Since only the intermediate temperature range was required to generate the Glover-Rowe parameters in this study, test temperatures of 35, 25, 15°C were used. After the test temperature achieved equilibrium, the G* and phase angle were determined over a range of frequencies (radians per second) shown in Table 7. The idea of utilizing the selected frequencies shown in Table 7 is that the range covers two decades of loading times, providing five data points per log decade of frequency tested.

Table 7 – Recommended Range of Frequencies for DSR Frequency Sweep Testing

Log Basis (radians/second)	Linear Basis (radians/second)
-1.0	0.100
-0.8	0.159
-0.6	0.251
-0.4	0.398
-0.2	0.631
0.0	1.00
0.2	1.59
0.4	2.51
0.6	3.98
0.8	6.31
1.0	10.0

The initial data was inspected for quality by plotting the results of G* vs phase angle (i.e. – Black Space). The objective of this plot is to enable gross errors in the data to be spotted and eliminated before developing the final master curve and it respective parameters. It should be noted that smooth curves may not always exist due to transitions that may occur in the material. However, most asphalt binders when tested in the linear range, without modifiers, generally have a smooth relationship in this plot. Curves or "feathers" in the Black Space plot are generally associated with lower quality testing or utilizing too fast of a loading frequency during testing.

The resultant Glover-Rowe Parameter (GRP) results are shown in Figure 19 for the Traditional approach while Figure 20 shows the NCHRP 9-59 project approach. In the Traditional GRP, only the 45% RAP with Soylei recovered asphalt binder was able to meet the < 180 kPa *Crack Warning* criteria after 20 hour PAV conditioning. After 40 hours of PAV conditioning, all of the recovered asphalt binders failed the < 600 kPa *Cracking On-set* criteria. The recovered asphalt binders from the 45% RAP mixture with the Soylei appears to provide superior cracking performance over the 40% RAP Control mixture after both 20 hour and 40 hour PAV conditioning.

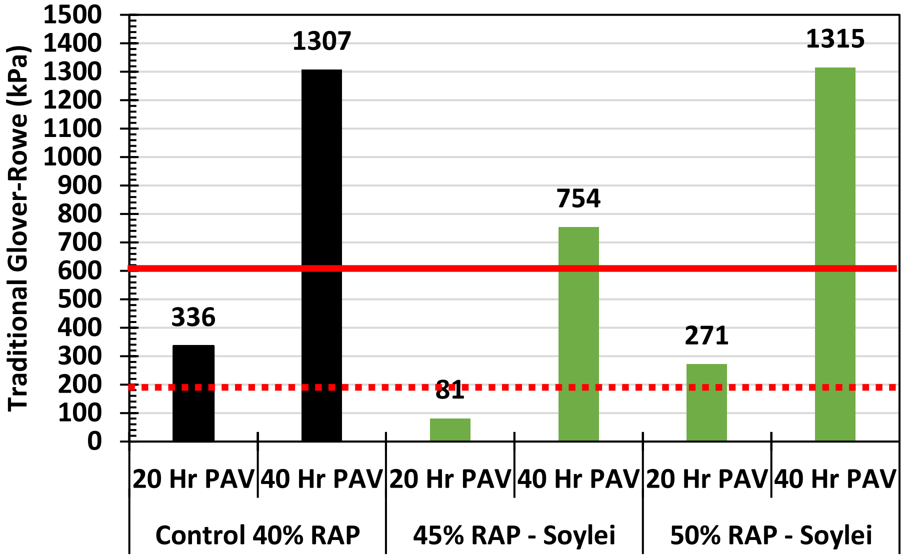


Figure 19 – Traditional Glover-Rowe Test Results

The NCHRP 9-59 Glover-Rowe results are shown in Figure 20. The trend in results is similar to those shown for the Traditional Glover-Rowe in Figure 19. Only the 45% RAP with Soylei recovered asphalt binder was able to meet the 5,000 kPa parameter at a test temperature of 25°C and 10 rad/sec. And like before, all three recovered asphalt binders were not able to meet the criteria of 8,000 kPa after 40 hours PAV conditioning.

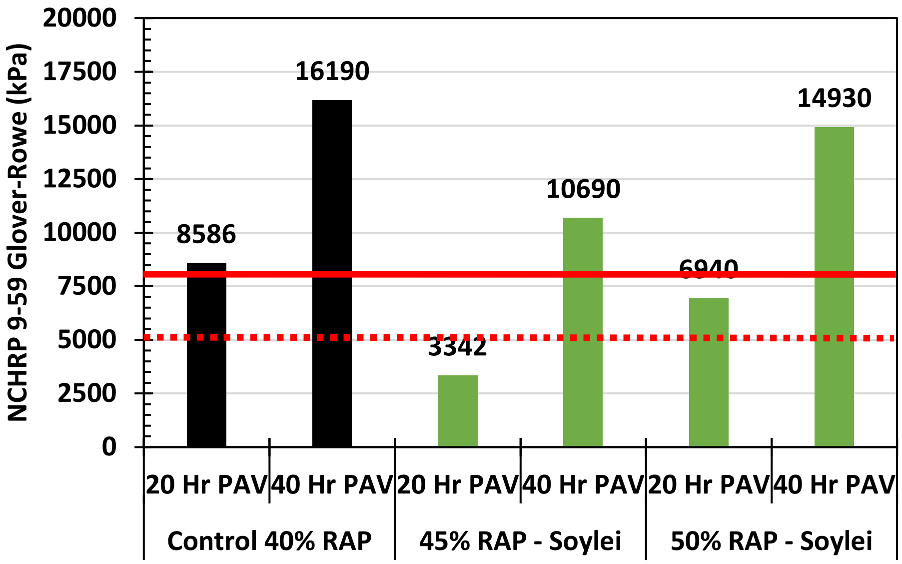


Figure 20 – NCHRP 9-59 Glover-Rowe Test Results

Figure 21 shows the ΔTc results for the 20 hour and 40 hour PAV conditioned recovered asphalt binders. Currently, nationwide criteria have yet to be established regarding a minimum ΔTc value, although most commonly used values suggest the following:

- 20 Hour PAV Conditioned: ≥ 2.5°C

- 40 Hour PAV Conditioned: ≥ 5.0ºC

At the 20 hour PAV conditioning, all of the recovered asphalt binders were within 1.2ºC of one another. The best performing recovered binder was from the 45% RAP with Soylei. A wider spread of results was found with the 40 hour PAV conditioned recovered binders with the range in results being 2.2ºC. None of the recovered asphalt binders would have met the requirement of -5.0ºC for the 40 hour PAV conditioning. It should be noted that recent research conducted by the Maryland State Highway Administration (MDSHA) has shown that the single operator allowable range of two results for ΔTc is 0.8ºC while multiple laboratory allowable range of results is 2.4ºC (Azari and Akisetty, 2024). Overall, the recovered asphalt binder from the high RAP contents with Soylei provides equal performance to the 40% RAP Control recovered asphalt binder at the identical conditioning level.

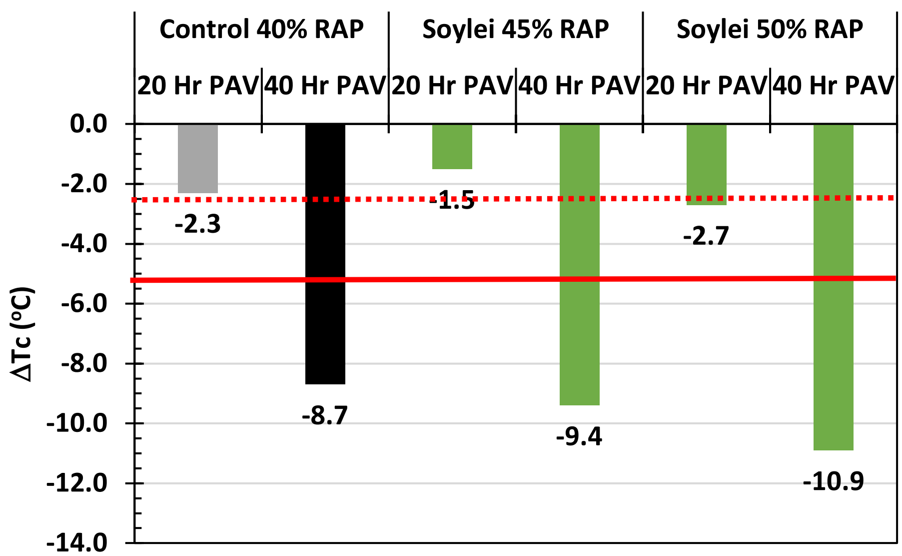


Figure 21 – ΔTc Test Results

The last asphalt binder analysis conducted was the NCHRP 9-60 approach that utilizes the BBR ΔTc and the ABCD device critical cracking temperature, Tcr. The analysis for the 20 hour PAV and 40 hour PAV are shown in Figures 22 and 23, respectively. Using the proposed Pass/Fail criteria proposed by NCHRP 9-60 project shows that after 20 hour PAV conditioning, only the recovered asphalt binder from the 45% RAP with Soylei mixture would have passed. All recovered binders conditioned for 40 hours in the PAV would not have met the proposed NCHRP 9-60 criteria.

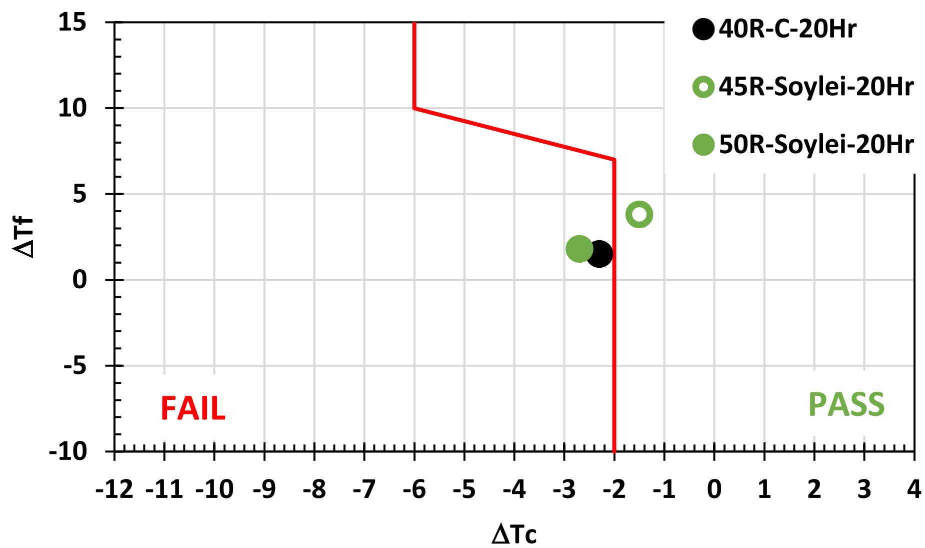


Figure 22 – NCHRP 9-60 ABCD Cracking Approach – 20 Hour PAV Conditioned

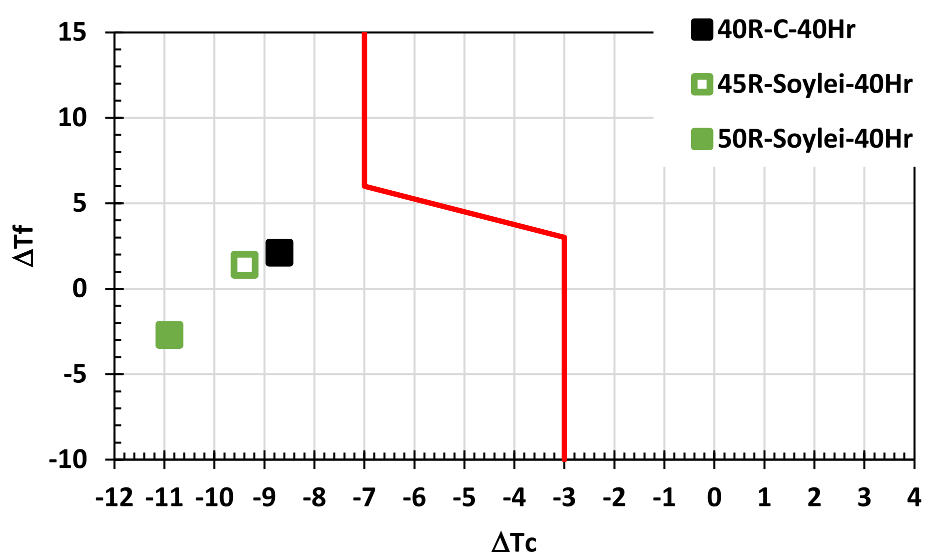


Figure 23 – NCHRP 9-60 ABCD Cracking Approach – 40 Hour PAV Conditioned

FINDINGS AND CONCLUSIONS

A field research study was conducted to evaluate the impact of the Soylei recycling aging with two different RAP contents (45% and 50%) on the stiffness, rutting, and cracking performance of asphalt mixtures with high percentages of recycled asphalt pavement (RAP). The recycling agent was blended in-line at the asphalt plant and used at the dosage rate of 1.8 lbs per ton of hot mix when incorporating into a 45% RAP mixture and 2.0 lbs per ton of hot mix for the 50% RAP mixture, respectively. Performance characterization was conducted on both sampled loose mix and recovered asphalt binder from the collected loose mix. The results of the study showed:

- RAP content can be confidently increased from 40% to 45% in New York City with the incorporation of Soylei recycling agent used in this study. Most of the characterization conducted showed that cracking resistance improved with the addition of the RA with minimal detrimental impact on the rutting resistance. Even better fatigue cracking performance may have been observed if the asphalt contents were the same for the 45 to 50% RAP mixtures (averaged 5.0%) as it was for the Control 40% RAP mixture (5.3%).
- Although the asphalt binder properties of the 50% RAP mixtures showed equal to better performance than the Control 40% RAP mix, mixture testing showed a decrease in fatigue performance at the 50% RAP level while providing equivalent rutting performance. Again, this may be attributed to the lower asphalt content in the 50% RAP mixtures when compared to the Control 40% RAP mixture. The 45% RAP with Soylei mixture performed the best throughout the study.
- Using the BMD performance criteria being evaluated by the NYSDOT, the 45% RAP mix with Soylei would have met both the rutting (HT-IDT Strength) and fatigue cracking (IDEAL-CT Index). The 40% RAP Control mix with no RA did not meet the requirements.

The study illustrates that higher RAP content asphalt mixtures can be confidently produced using the Soylei recycling agent that obtains similar to better asphalt binder and mixture performance than the current 40% RAP asphalt mixtures used by the New York City Department of Transportation (NYCDOT).

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