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Initial Exploration of Reduced Gyrations for Arkansas Asphalt Mix Designs

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Initial Exploration of Reduced Gyrations for Arkansas Asphalt Mix Designs

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by

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Abstract

The most common distresses in asphalt pavement are cracking and rutting. These distresses plague almost every asphalt roadway in the world. Finding ways to mitigate these distresses will improve the performance of asphalt pavements. One way to combat the premature cracking of asphalt in the field is to ensure there is enough asphalt binder within the mix. This will help the flexibility of the asphalt undergoing large loads and deformation. However, it is widely accepted that mix designs across the country lack the appropriate amount of asphalt binder in their mixes, causing dry mix designs that are subject to cracking before their intended design life has expired. Twenty-seven states across the United States have lowered the number of design gyration requirements to encourage the design of mixes with a slightly higher binder content. Arkansas is not one of those states. Therefore, this research investigating the performance of lower design gyrations for Arkansas asphalt mix designs is the initial exploration and proof of concept for Arkansas. By comparing the cracking tolerance index (CT_{Index}) of field cores, plant-mixed lab-compacted, and lab-mixed lab-compacted samples, numerous conclusions can be drawn, and further research can be developed. This study tested field cores and plant mixed lab compacted samples for four different mix designs, two with recycled asphalt concrete (RAP) and two without, from four different asphalt plants in Arkansas. Two of those mix designs, one with RAP and one without, were selected to be evaluated using lab-mixed lab-compacted samples as well. The results of this study showed that field cores, in general, performed with a higher cracking tolerance over plant-mixed lab-compacted and lab-mixed lab-compacted samples. The lab-mixed lab-compacted samples outperformed the plant-mixed lab-compacted samples in both instances they were tested, verified by a paired t-test. This study

serves well as a basis for future studies on the reduction of gyrations for Arkansas asphalt mix designs.

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Literature Review

From 1987 to 1993, the Strategic Highway Research Program (SHRP) embarked on an intense study that resulted in the Superior Performing Pavement, known as Superpave. The Superpave system stemmed from the volumetric testing and analysis found within the Hveem and Marshall mix design standards. The SHRP study defined three compactive efforts for the Superpave mix design procedure: N_{ini}, the initial compactive effort, N_{des}, the design compactive effort, and N_{max}, the maximum compactive effort. The parameter N_{des} was described as "a function of average design air temperature and [equivalent single axle loads (ESALs)], and [varied] from 68 to 172." For each compacted asphalt specimen, the volumetric properties, air voids, voids in mineral aggregate (VMA), voids filled with asphalt (VFA), and density, are determined at N_{des} gyrations (Kennedy et al., 1994). Table 1 recreates the original SHRP table outlining the design gyrations.

Table 1. Design Gyration Values (Kennedy et al., 1994).

Traffic (ESALs)	**Average Design Air Temperature (℃)**			
	< 39	**39-41**	**41-43**	**43-45**
$< 3 \times 10^5$	68	74	78	82
$< 1 \times 10^6$	76	83	88	93
$< 3 \times 10^6$	86	95	100	105
$< 1 \times 10^7$	96	106	113	119
$< 3 \times 10^7$	109	121	128	135
$< 1 \times 10^8$	126	139	146	153
$> 1 \times 10^8$	143	158	165	172

Since the initial development of Superpave, the table has been consolidated to include only four gyration levels for N_{ini}, N_{des}, and N_{max}. Although, when this consolidation took place, there was not an experiment conducted to validate the selected values (Aguiar-Moya et al. 2007). Arkansas currently follows the AASHTO Standard Practice for Superpave Volumetric Design for Asphalt Mixtures R 35-17 and utilizes four compaction levels for N_{des}, outlined in Table 2. The main concern surrounding the number of design gyrations is that the existing standard may be too high. While mixes designed with higher N_{des} values have a lower potential to rut, the compactibility in the field can be difficult and the mixes have a low asphalt binder content, leading to potential durability issues in the future.

Table 2. Recreation of AASHTO R 35-17 Superpave Gyratory Compaction Table

Design ESALs (million)	Compaction Parameters		
	$N_{initial}$	N_{design}	N_{max}
< 0.3	6	50	75
$0.3\ to < 3$	7	75	115
$3\ to < 30$	8	100	160
≥ 30	9	125	205

The reduction of the N_{des} gyration levels for Superpave mixture designs has been successfully passed in 27 states across the United States (Tran et al. 2019). The recent report from the National Center for Asphalt Technology (NCAT) and Heritage Research Group summarized the changes made with several of the 27 states lowering their design gyration levels to a range between 50 and 75 gyrations. With this change in gyration levels, 18 of the states stated they also increased their asphalt content and 8 lowered design air voids (Tran et al. 2019). Please refer to Appendix A, Table 7. and Table 8., for a summary of state changes to design gyrations and VMA levels.

In 2000, when Virginia's Department of Transportation (VDOT) first implemented Superpave in their state, they decided on an N_{des} value of 65 for surface mixes and 75 for all other mixtures. However, in 2002, VDOT subsequently lowered the requirement to 65 design gyrations for all mix designs in an effort to "provide higher binder contents" and "increase durability" (Bowers et al. 2018). With further testing, Virginia settled on 50 design gyrations for surface mixes and 65 design gyrations for intermediate and base layer mixes.

Georgia was another state to successfully pass a reduction in design gyrations. A validation of the design gyration compaction levels for Georgia was conducted and resulted in the Georgia Department of Transportation increasing "the minimum VMA requirements for all Superpave mixtures by 1.0%" and settling on 65 design gyrations across the board. It was also found that the locking point for Georgia's mixes was around 69 gyrations. For this study, the locking point was defined as the point at which the sample's height "remained the same for two, three, and four consecutive gyrations," where further gyrations could lead to degradation of the aggregates (Watson et al. 2008).

The main goal in reducing the design gyration levels in mixture design is to improve the durability and compactibility of current Superpave mix designs. With a reduction of gyration levels, an increase in asphalt content should be expected. An increase in asphalt content leads to an increased concern for rutting potential. However, in Virginia and Georgia, it was discovered that even with a VMA increase of 1%, rutting issues did not occur (Maupin 2003, Watson et al. 2008).

Research Topic and Background

The need for more durable and compactible mixture designs in Arkansas is relevant. A task force meeting with several contractors across Arkansas and the Arkansas Asphalt Pavement Association (AAPA) was held on January 14th, 2021. It was apparent, in that meeting, contractors across the state were willing to provide aggregate to perform the necessary testing and research of new design gyration levels in Arkansas. While this is the first step in exploring a possible reduction in gyration level for Arkansas, it is an important step forward towards making a formal recommendation to the Arkansas Department of Transportation (ARDOT) for the modification of standards.

Contractors present during the January 14th meeting expressed interest toward a total of two design gyration levels instead of the current number of four. The meeting concluded with Arkansas contractors suggesting design gyration levels should be 85 for higher traffic highways and 60 for lower traffic roadways. However, gyration levels at 75 and 50 were determined to have merit as well. The gap of 25 gyrations between the upper and lower levels is imperative to see viable change in a mix design.

Four ARDOT projects were granted permission to implement a special provision to reduce the design gyrations to 75 for an existing mix on a job and tested in this research project. The contractors that requested to run this provision included:

- Atlas Asphalt, Inc. Batesville, #A50005, no VMA change
- Atlas Asphalt, Inc. Jonesboro, #A10006, +1.0% VMA
- Delta Asphalt of Ark., Inc. Judsonia, #A50008, +0.5% VMA
- Jet Asphalt & Rock Co., Inc. Bearden, #A70005, no VMA change

All mix designs were modified to include the special provision titled “Design Gyrations and Percent Air Voids in Mineral Aggregate (VMA%) for ACHM Surface Mix Designs” with or without VMA change with design gyrations reduced to 75. The goal of this research was to compare the cracking performance of plant-mixed plant-compacted samples (field cores), plant-mixed lab-compacted (PMLC) samples, and lab-mixed lab-compacted (LMLC) samples to determine if the mixes offer equal, or better, cracking tolerance.

Materials and Methods

During the summer and fall of 2021 materials were gathered from all four asphalt plants that were producing the lower gyration mix designs for the aforementioned ARDOT jobs. Each plant provided approximately 25-30 buckets of the necessary aggregates, four to five gallons of the appropriate binder, and truck sampled plant mix that was traveling to the job site. After the job was completed each of the resident engineers for the jobs supplied field cores to be tested alongside the lab compacted samples; the number of field cores varied for each job ranging from three to fourteen. The field cores were all drilled from lots within the job site boundaries with a 150mm (approximately 6-inch) coring drill, excluding the Delta Asphalt field cores which were obtained with a 100mm (approximately 4-inch) coring drill. All field cores obtained were low gyration mixes. All aggregate was dried, fractionated and stored in five-gallon buckets. The binder was stored in a temperature-controlled laboratory and the plant mix was stored within brown paper sacks in the same temperature-controlled lab area.

To begin processing the plant mix, each sample was removed from the brown sacks and placed into a lab pan. The plant mix was then reheated to the appropriate compaction temperature that was specified on the respective mix design. After thorough and even heating, the mix was then reduced using the quartering method outlined in the AASHTO Standard R47-

19, Reducing Samples of Asphalt Mixtures to Testing Size. Samples tested in the lab are to reach air voids of 7 ± 0.5%. To target this, a back calculation was done using the bulk specific gravity of the mix and the intended volume of the sample. Since the bulk specific gravity (G_{mb}) varies with each mix design, the back calculation was completed for each of the four mixes. The target weights were generally around 2500g which would provide samples containing the desired air voids of 6.5-7.5%. Maximum specific gravity samples were also obtained and verified.

The LMLC samples were prepared following the AASHTO Standard T 312-19, Preparing and Determining the Density of Asphalt Mixture Specimens by Means of the Superpave Gyratory Compactor for two out of the four sites. The lab mix was then conditioned following the short-term conditioning protocol for mixture mechanical property testing found in AASHTO R 30-02 Mixture Conditioning of Hot Mix Asphalt (HMA). This aging protocol was recommended by Dr. Elvis A. Castillo-Camarena and Dr. Hall to mimic the aging process that occurs during construction, this would allow for a more accurate comparison of the field cores, PMLC samples, and LMLC samples (Castillo-Camarena & Hall 2021).

The PMLC and LMLC samples were then individually compacted generally following the AASHTO Standard T 312-19. However, these samples were prepped for the Indirect Tensile Asphalt Cracking Test, known as the IDEAL-CT. Therefore, it was necessary to deviate from the standard and compact to a height of 62 ± 1mm. After the samples were compacted, they were labeled and set aside to cool. The samples were then tested for air voids and bulk specific gravity using the AASHTO Standard T 331-13, Bulk Specific Gravity (G_{mb}) Density of Compacted Asphalt Mixtures Using Automatic Vacuum Sealing Method. The G_{mm} samples obtained were also tested to validate the G_{mm} stated on the mix design for each job, these samples were tested following the AASHTO Standard T 209-20, Theoretical Maximum Specific Gravity (G_{mm}) and Density of Asphalt Mixtures.

The PMLC and LMLC samples were conditioned with a target conditioning temperature of 25 ± 1℃ in a water bath capable of maintaining a steady temperature and then evaluated following the ASTM Standard D8225-19, Determination of Cracking Tolerance Index of Asphalt Mixture Using the Indirect Tensile Cracking Test at Intermediate Temperature. The IDEAL-CT test was conducted during this research because of its practical application in the laboratory. This test requires no cutting and can be performed on a Marshall load frame. The sample is placed in the load frame and the load is applied vertically. As the load is applied, a linear variable differential transformer (LVDT) measures the load-line displacement (LLD) and a data acquisition system is used to collect the load, time, and LLD with at least 40 sampling points/second. See Figure 1. for a visual on the Marshall load frame and specimen placement. In a previous study, a cracking tolerance index (CT_{Index}) of greater than 50 was proposed to the Arkansas Department of Transportation as a threshold for mix design acceptance and production (Castillo-Camarena & Hall 2021). See Appendix B for an explanation of parameters for the IDEAL-CT test.

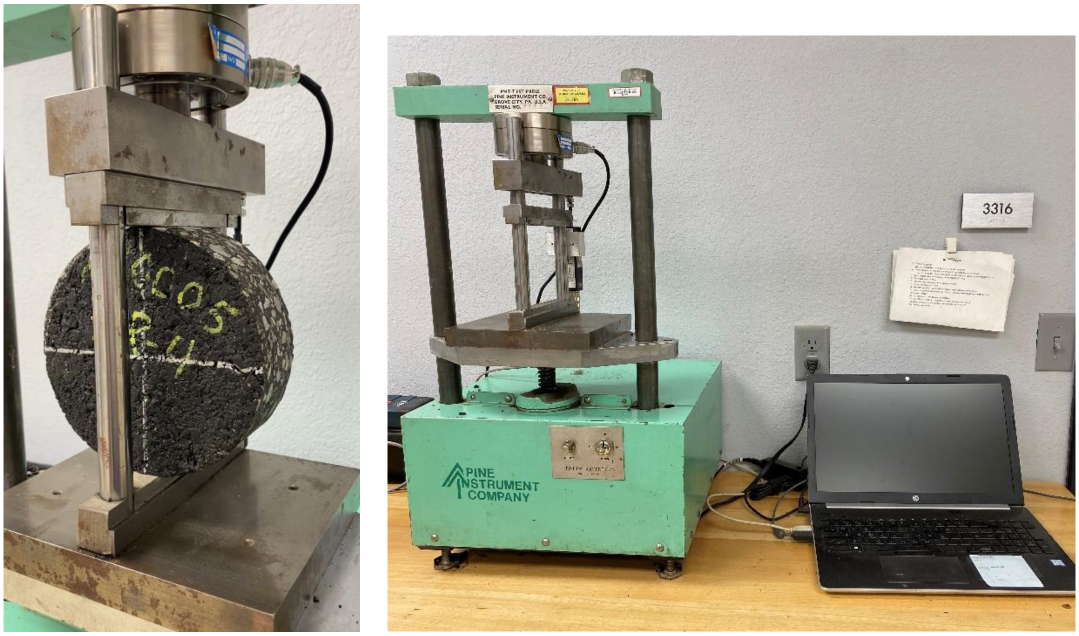


Figure 1. Marshall Testing Frame and IDEAL-CT Lab Equipment

After testing and collecting the data, the data file was uploaded into a MATLAB program developed by Dr. Elvis A. Castillo-Camarena at the University of Arkansas. This program allows for the input of the diameter and thickness of the tested specimen and calculates the CT_{Index}. This test was done for all field cores, PMLC, and LMLC samples regardless of the specimen's air voids (AV). This allowed for the comparison of the different types of samples at a variety of air void values.

Results and Discussion

After testing, results for all four sites were compiled. Each site had results for field cores and PMLC samples, additionally, LMLC samples were prepared and tested for Atlas Batesville and Atlas Jonesboro.

Delta Asphalt field cores provided CT_{Index} values ranging from 63 to 177 with a mean value of 109, where the PMLC samples ranged from 11 to 26 with a mean value of 23. Summary statistics for the Delta Asphalt field cores and PMLC samples can be found in Table 3.

Table 3. Summary Statistics for Delta Asphalt of Ark., Inc. out of Judsonia.

	CT_{Index}	
	Field Cores	**Plant-Mixed Lab-Compacted**
Mean	109	23
Median	100	24
Standard Deviation	36.5	6.8
Range	114	25
Minimum	63	11
Maximum	177	36

The data shows drastically differing performance for the field cores and PMLC samples, (see also Figure 2.). The Delta Asphalt field cores all performed with CT_{Index} values above the $CT_{Index} > 50$ threshold for Arkansas mix designs at a very wide range of air void values, indicating the mix in the construction mat is preforming at a higher level than the mix in the lab. All of the PMLC samples fell below the threshold. This difference could be attributed to the reheating and compactive efforts the PMLC samples experience in the lab. It is also pertinent to note that this mix design contained approximately 25% recycled asphalt pavement (RAP).

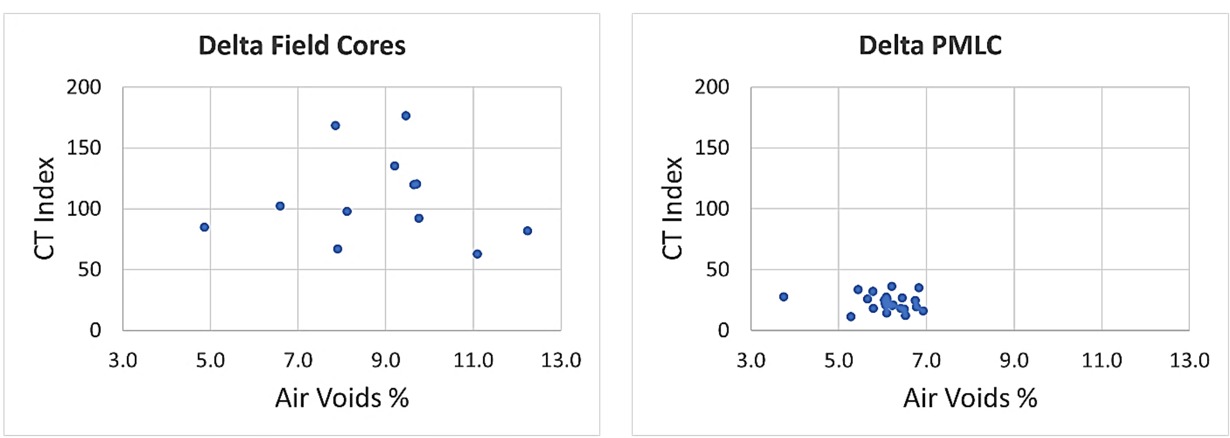


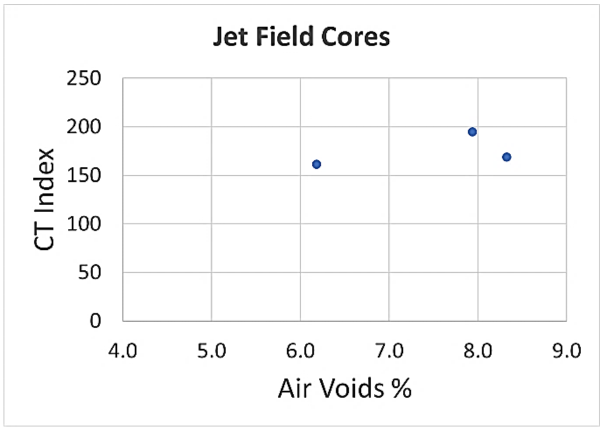
Figure 2. Delta Asphalt of Ark., Inc. Field Core and Plant-Mixed Lab-Compacted Results

The mix design from Jet Asphalt out of Bearden, AR was evaluated using field cores and PMLC samples also. The field cores for this site provided a mean value of 175 with a range from 161 to 195. The field core data was limited, due to only having 3 samples. The PMLC samples had a mean value of 57 with a range from 17 to 114. Table 4. provides a concise summary of the statistics for the Jet Asphalt site.

Table 4. Summary Statistics for Jet Asphalt out of Bearden.

	CT_{Index}	
	Field Cores	**Plant-Mixed Lab-Compacted**
Mean	175	57
Median	169	57
Standard Deviation	17.6	24.7
Range	34	96
Minimum	161	17
Maximum	195	114

It is evident that the Jet Asphalt field cores performed better than the PMLC specimens. Again, this could be contributed to the varying conditions these samples experienced and should be expected. However, as you can see in Figure 3., the majority of the PMLC preformed above the threshold recommended to ARDOT, $CT_{Index} > 50$, and the three field cores tested were consistent across a range of air voids.



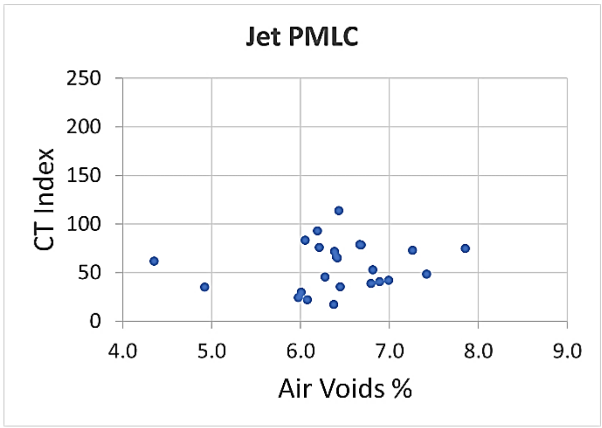


Figure 3. Jet Asphalt Field Core and Plant-Mixed Lab-Compacted Results

The next site that was evaluated was Atlas Asphalt, Inc. out of Batesville, AR. This site was one of the two sites selected to be evaluated using all three sampling methods, field cores, PMLC, and LMLC. It was apparent after testing the field cores and PMLC samples from this site that this mix design was the highest performing in regard to CT_{Index} values. Therefore, LMLC samples were prepared and tested to compare to the field cores and PMLC sample results.

The Atlas Batesville field cores CT_{Index} values provided a range between 62 and 432 with a mean value of 245. The PMLC CT_{Index} values ranged from 31 to 110 and provided a mean of 59. CT_{Index} values for the LMLC samples ranged from 74 to 109 with a mean of 90. Table 5. provides a summary of the statistics for the field cores, PMLC, and LMLC samples for Atlas Asphalt Batesville.

Table 5. Summary Statistics for Atlas Asphalt, Inc. out of Batesville.

	CT_{Index}		
	Field Cores	**Plant-Mixed Lab-Compacted**	**Lab-Mixed Lab-Compacted**
Mean	245	59	90
Median	236	59	88
Standard Deviation	108.5	18.4	11.2
Range	370	79	35
Minimum	62	31	74
Maximum	432	110	109

The results for the Atlas Asphalt Batesville lower gyration mix design are promising since all three types of samples tested provided a mean greater than the aforementioned lower limit of $CT_{Index} > 50$ for Arkansas asphalt mix designs. As expected from previous results, the field cores performed better than the PMLC, see Figure 4.

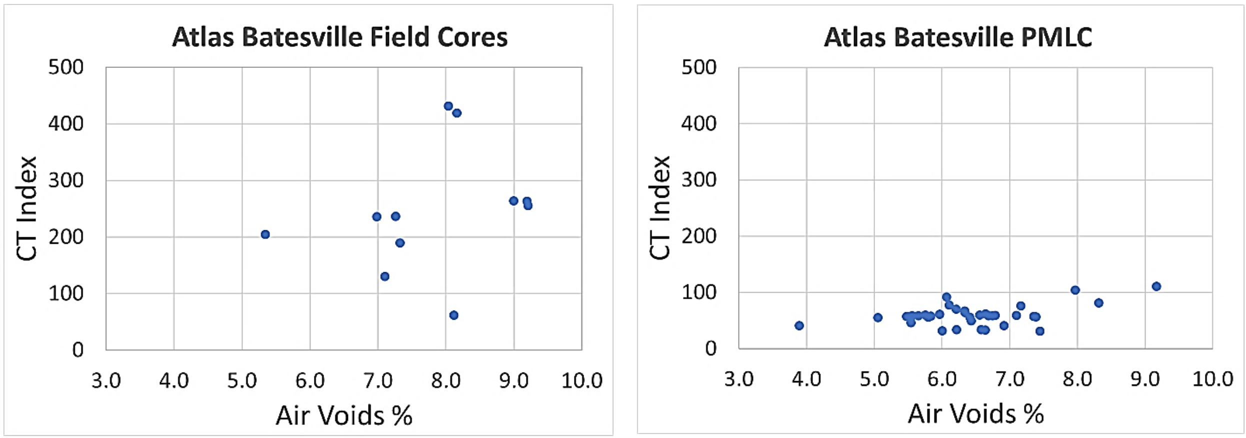


Figure 4. Atlas Asphalt Batesville Field Core and Plant-Mixed Lab-Compacted Results

The averages for the Atlas Batesville PMLC and LMLC weren't drastically different at 59 and 90, respectively. Therefore, a paired t-test for means was performed to determine if there was a statistically significant difference in the means for both groups. A paired two tailed t-test was chosen since the two groups originated from the same mix design but were prepared using separate methods. A t-test assumes the following hypotheses: H_0: The difference between the group means is zero. H_a: The difference between the group means is different than zero. First, a simple random sample of the PMLC samples was conducted which provided nine CT_{Index} values to compare to the nine LMLC samples obtained. The t-test was conducted using a significance value of $\alpha = 0.05$ and resulted in a t-value = 4.9 and a p-value = 0.0011. With t-critical = 2.306, we can safely assume that the difference in the means for the LMLC and PMLC is statistically significant. Figure 5. shows a comparison of the raw data for the complete populations of the LMLC and PMLC samples.

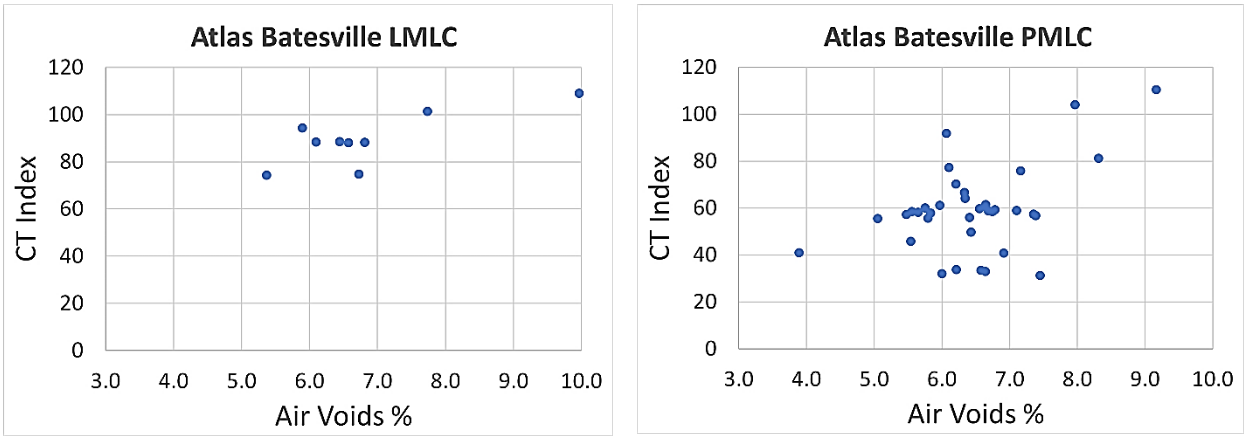


Figure 5. Atlas Asphalt Batesville Lab-Mixed Lab-Compacted and Plant-Mixed Lab-Compacted Results

A similar approach was taken for the mix design from Atlas Asphalt, Inc. out of Jonesboro. Field cores, LMLC, and PMLC samples were prepared and tested. This mix was chosen because it was the worst performing mix in regard to CT_{Index} values for the PMLC samples. This mix also contained approximately 15% RAP.

The CT_{Index} values for Atlas Jonesboro's field cores ranged between 102 and 261 with a mean value of 151. The PMLC CT_{Index} values ranged from 7 to 35 and provided a mean of 21. The LMLC CT_{Index} values ranged from 52 to 119 with a mean value of 77. Table 6. provides a statistics summary for the field cores, PMLC, and LMLC samples for Atlas Asphalt Jonesboro.

Table 6. Summary Statistics for Atlas Asphalt, Inc. out of Jonesboro.

	CT_{Index}		
	Field Cores	**Plant-Mixed Lab-Compacted**	**Lab-Mixed Lab-Compacted**
Mean	151	21	77
Median	142	21	76
Standard Deviation	44.1	8.8	18.7
Range	159	28	66
Minimum	102	7	52
Maximum	261	35	119

As previously seen, the field cores performed with higher CT_{Index} values than the PMLC samples. The PMLC samples for this mix did not produce any CT_{Index} values greater than 35 which is below the > 50 threshold for the CT_{Index} value. Figure 6. depicts the raw data for Atlas Jonesboro's field cores and PMLC samples.

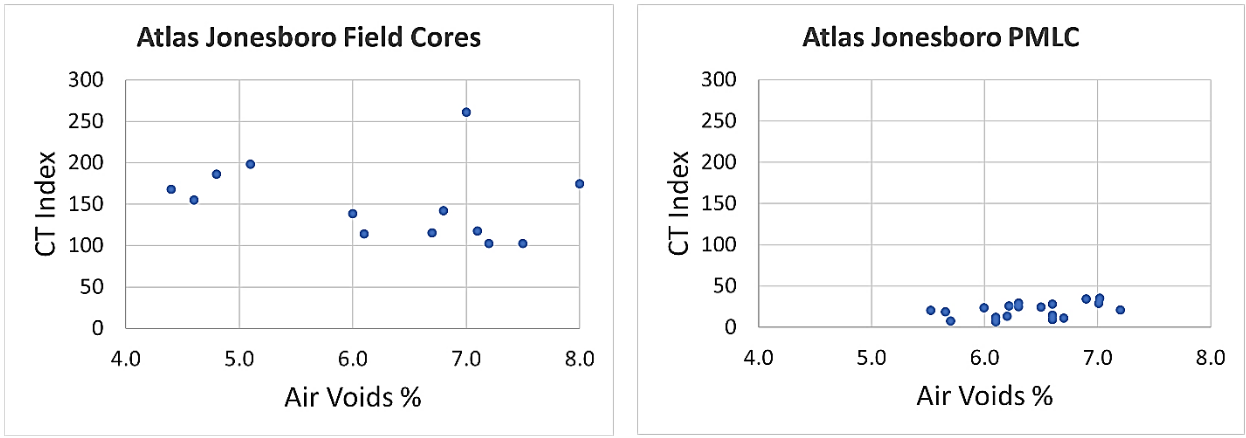


Figure 6. Atlas Asphalt Jonesboro Field Core and Plant-Mixed Lab-Compacted Results

A paired t-test for means was performed again to determine if there was as statistically significant difference in the means for the PMLC and LMLC groups for Atlas Jonesboro. The two tailed t-test assumed the same hypotheses as before, and a simple random sample of the PMLC samples was conducted to provide ten CT_{Index}values to compare to the ten LMLC samples obtained. The t-test was conducted using a significance value of $\alpha = 0.05$ and resulted in a t-value = 10.2 and a p-value = 2.95E-6. With t-critical = 2.262, we can safely assume that the difference of the two means is statistically significant. Figure 7. shows the complete populations of the LMLC and PMLC samples for Atlas Asphalt Jonesboro.

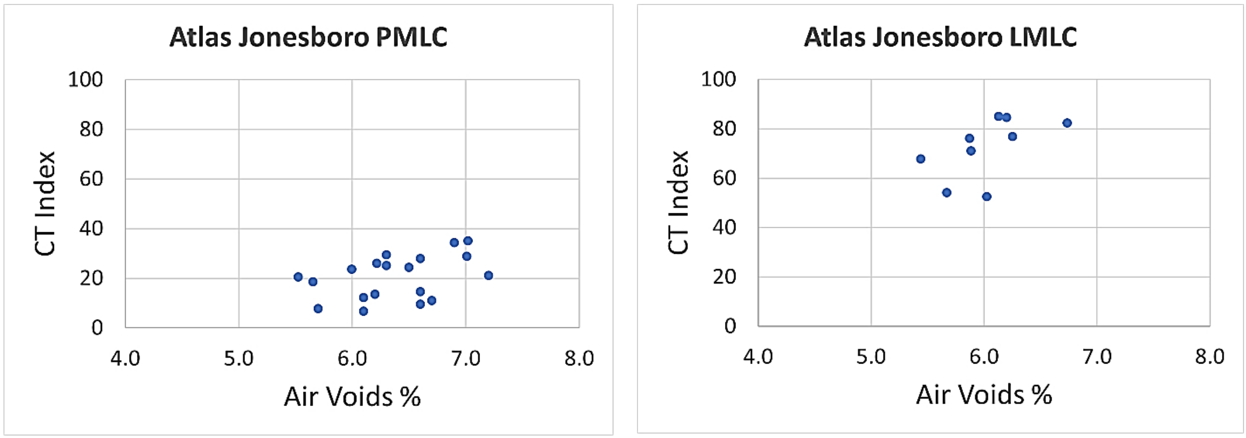


Figure 7. Atlas Asphalt Jonesboro Lab-Mixed Lab-Compacted and Plant-Mixed Lab-Compacted Results

Aside from comparing the means for each sample method and site location, it is pertinent to examine the graph produced by conducting the IDEAL-CT test. A general observation made during the testing process was that samples with a higher CT_{Index} had a lower peak load while samples with a lower CT_{Index} had a higher peak load. This seems counter intuitive, however when looking at the graphs the reason for this becomes apparent. See Figure 8. for a comparison of Atlas Jonesboro's field core 2-2,CT_{Index} = 116, and PMLC sample #19, CT_{Index} = 29.

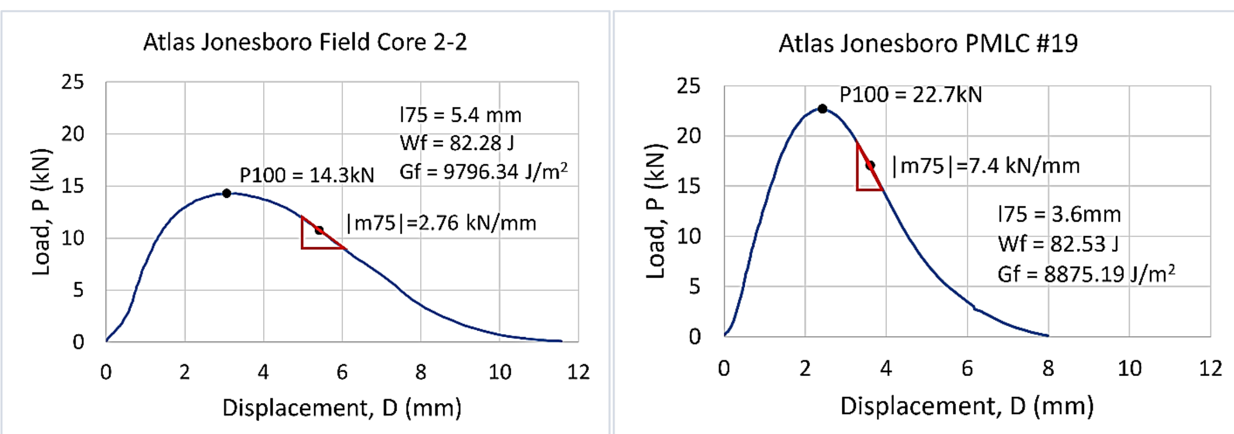


Figure 8. Atlas Asphalt Jonesboro Field Core 2-2 and Plant-Mixed Lab-Compacted #19 IDEAL-CT Test Results

The graphs in Figure 8. depict a load versus displacement curve. The load represents the load (kN) that the sample is experiencing at the corresponding load-line displacement (mm). The peak load, labeled P_{100}, is shown at the top of the curves and the slope at 75% of the peak load is show as |m75|. The higher the slope at that point the faster the sample is failing. It is also apparent the slope on the backside of the curve indicates the type of failure the sample is experiencing; a steeper slope would suggest a more brittle failure of the sample. It is evident from the graphs in Figure 8. that the PMLC sample reached a higher peak load, however the sample did not sustain that load. While the field core did not reach as high of a peak load, it maintained the load longer, creating a larger area under the curve and subsequently producing a higher CT_{Index}.

Conclusion

This project presented an initial exploration into the reduction of gyrations for Arkansas asphalt mix designs. The results obtained were from four different mix design placed in four different locations. Two of the mix designs, Delta Asphalt of Ark., Inc. out of Judsonia and Atlas Asphalt, Inc. out of Jonesboro, contained RAP while the other two did not. The samples tested for each mix included field cores and PMLC samples. Atlas Asphalt, Inc. Batesville and Jonesboro mixes were selected to be evaluated using LMLC samples as well.

Conclusions and observations stemming from this work include:

- The CT_{Index} for field cores were higher than PMLC and LMLC for all mix designs tested.
    - For future considerations regarding cracking resistance in the mixture design process, with mixtures showing a higher cracking resistance in the field, a conservative acceptance threshold for the CT_{Index} may is not necessary.

- The CT_{Index} for mix designs including RAP with a VMA increase were lower than the CT_{Index} for mixes not containing RAP with no change in VMA. It is recommended that determining the effect of the inclusion of RAP on the cracking resistance of the mix (particularly when more than 15 percent of the aggregate blend) be a key element of subsequent research efforts.
- The difference in means for Atlas Batesville and Atlas Jonesboro PMLC and LMLC samples are each statistically significant.
    - We can conclude that the LMLC samples did have higher CT_{Index} values than the PMLC samples for both Atlas Batesville and Atlas Jonesboro. The reheating of the plant mix before compaction could be the cause for the lower performance.
- All LMLC samples provided values of $CT_{Index} > 50$, which is above the threshold previously proposed to ARDOT.
- Test specimens in this study that exhibited a higher peak load did not demonstrate a higher CT_{Index}.
    - The CT_{Index} attempts to capture both crack *initiation* (related to the peak load) and crack *propagation* (related to the post-peak slope). For example, a high peak load with subsequently lower CT_{Index} suggests a more 'brittle' failure; conversely, a lower peak load with subsequently higher CT_{Index} suggests a more 'ductile' failure. Understanding this behavior may help the mix designer when selecting materials, binder content, and volumetric properties.

Summary

This study shows the potential for reduced gyration mix design in the state of Arkansas and the validity of simplifying specifications to include lower N_{des} levels. While this study only explored a reduction from 100 to 75 design gyrations, a greater reduction to 50 design gyrations also has merit and could produce greater cracking tolerance. It would also be advantageous to evaluate the cracking tolerance of mix designs with an increased VMA that do not include recycled asphalt pavement. See Appendix C for an explanation on the relationship between reduced gyrations, binder content, VMA, and locking point. While further testing on the topic of reduced design gyrations for Arkansas asphalt mix designs is already underway, this project provides valuable observations for future investigations.

Many agencies across the United States have adopted a reduced gyration specification to allow for lower N_{des} values for higher traffic projects, some concluding that one N_{des} level across the board is sufficient. It is important for Arkansas to make strides toward developing mixture designs that are more buildable and resistant to premature cracking in the field. To do this Arkansas needs to reevaluate the standards surrounding the number of design gyrations. Adding additional asphalt to a mix, and lowering design gyrations, has proven effective in more than half of the United States. A reduction of gyrations will most likely lead to an increase in asphalt binder, and this may lead to a cost concern for contractors. However, this should not impede with the forward progress of this research. Prices may increase, but with more crack resistant mix designs, Arkansas roadways will last longer and in turn save the state money. The reduction of the design gyration levels in Arkansas is long overdue. With this research, Arkansas can make an initial step forward in producing more durable mix designs for the future.

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Appendix A

Table 7. Highway Agencies That Have Lowered Design Gyration Levels (Tran et al. 2019)

Design ESALs (Million)	**Design ESALs (Million)**			
	< 0.3	$0.3\ to < 3$	$3\ to < 30$	≥ 30
AASHTO R 35	50	75	100	125
AL	60	60	60	60
CA	85	85	85	85
CO	50/75	75/100	75/100	125
CT	50	75	100	100
DE	75	75	75	75
GA	65	65	65	65
IA	ST (standard traffic): 50 HT (high traffic): 75 VT (very high traffic): 95			
IL	30	50-70	70-90	90
MD	50	65	80	100
ME	50	50	75	75
MO	50	75	80/100	125
MT	75	75	75	75
NC	50	50	65	100
ND	65/75	65/75	75	75
NE	SPS (shoulder mix): 40 SPR (high recycle mix): 65 SPH (heavy truck applications): 95			
NJ	50	50	50	50
OH	50	50	65	65
OK	50	50	65	80
OR	65	80	80/100	100
PA	50	75	100	100
RI	50	50	50	50
SC	50	75	75	75
SD	50	60	80	80
UT	50	75	75/100	75/100
VA	Surface-layer mix: 50 Intermediate-layer mix: 65 Base-layer mix: 65			
VT	50	65	80	80
WV	50	65	80	100

Table 8. Highway Agencies That Have Increase Minimum VMA (Tran et al. 2019)

NMAS	Minimum VMA (%)			
	19mm	**12.5mm**	**9.5mm**	**4.75mm**
AASHTO M 323	**13.0**	**14.0**	**15.0**	**16.0**
AL	13.5	14.5	15.5	16.5
CA	13.5	14.5	15.5	16.5
CO	13.6-13.8	14.6-14.8	15.6-16.9	N/A
DE	13.5	14.5	15.5	16.5
GA	14.0	15.0	16.0	N/A
IL	13.5	N/A	15.0	18.5
MA	14.0	15.0	16.0	17.0
ME	14.0	15.0	16.0	16.0
MT	13.0	14.5	15.5	N/A
NC	13.5	N/A	15.5	16.0
PA	13.5	14.5	15.5	16.0
RI	14.5	15.5	16.5	17.5
SC	13.5	14.5	15.5	17.5
SD	N/A	14.5	N/A	N/A
VA	13.0	15.0	16.0	16.5
VT	14.5	15.5	16.5	17.5
WI	13.0	14/14.5	15/15.5	16.0
WV	13.5	14.5	15.5	16.5

Appendix B

While the analysis for the IDEAL-CT test is relatively easy, understanding the relationships between the measured parameters is crucial. Figure 9. shows a comparison between Atlas Jonesboro's field core 2-2 and PMLC #19. It is evident that the graphs produced by these two samples in the Marshall testing frame are very different. Therefore, the CT_{Index} values differ as well, the field core produced a CT_{Index} = 116 and the PMLC sample produced a CT_{Index} = 29.

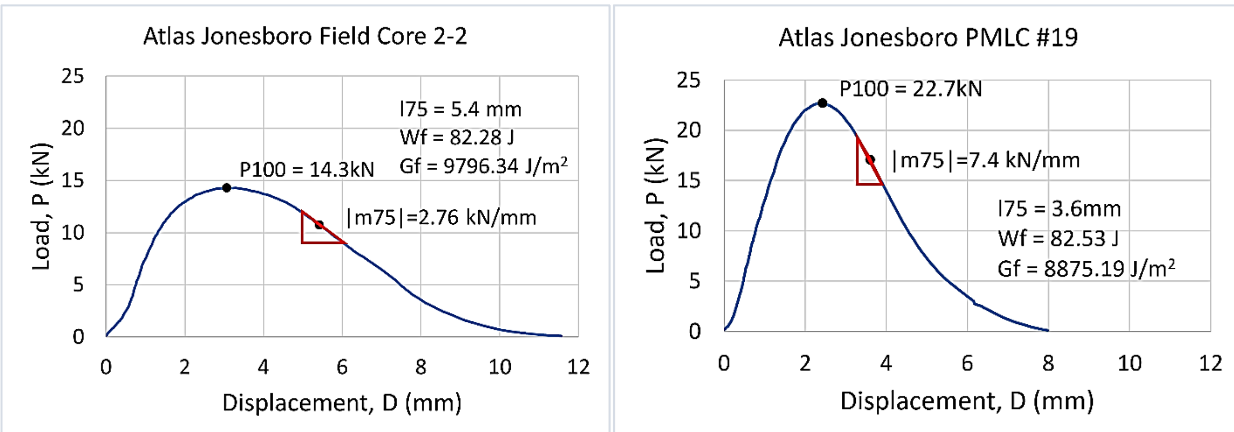

Figure 9. Atlas Asphalt Jonesboro Field Core 2-2 and Plant-Mixed Lab-Compacted #19 IDEAL-CT Test Results

To understand the difference in performance of these two samples it is crucial to examine how the individual parameters relate within the equation for the CT_{Index} and failure energy.

$$G_f = \frac{W_f}{D \times t} \times 10^6$$

$$CT_{Index} = \frac{t}{62} \times \frac{l_{75}}{D} \times \frac{G_f}{|m_{75}|} \times 10^6$$

where:

G_f = failure energy (Joules/m2),

W_f = work of failure, the area under the curve (Joules),

D = specimen diameter (mm),

t = specimen thickness (mm),

CT_{Index} = cracking tolerance index,

l_{75} = displacement at 75% the peak load after the peak (mm), and

$|m_{75}|$ = absolute value of the post-peak slope (N/m)

From the first equation it is apparent that as the area under the curve, W_f, increases, the total failure energy, G_f, increases as well. The CT_{Index} equation is mainly influenced by the total failure energy and the post-peak slope. Therefore, as the area under the curve increases, and the slope decreases, the CT_{Index} increases. Another important relationship that influences the CT_{Index} is within the second term of its equation. This term shows that the higher the displacement at 75% of the peak load, the higher resistance the sample has to the load, therefore, the sample is more crack resist. The first term of the equation is a simple correction factor for any samples that do not have a 62mm thickness.

Appendix C

The main goal of reducing gyrations in an asphalt mix design is to increase the binder content of the mix, making the mix more durable and crack resistant. However, only reducing the gyration level does not inherently solve the problem of a dry mix. Since binder is the most expensive part of the mix, mix designers will adjust the gradation of the mix to minimize the voids in mineral aggregate (VMA) and therefore minimizing the optimum binder content. This defeats the purpose of lowering the gyration levels in the first place.

One approach to combat this issue is increasing the minimum VMA requirement for mix designs alongside the reduction of gyrations. The idea behind this approach is that with an increase in VMA you are inherently increasing the binder content. For example, if air voids are at 5% and VMA 14% that leaves 9% of the volume left for binder. However, if the VMA is increased to 15% there is now 10% potential volume left for the binder. While this seems like an easy solution, the concern surrounding this approach is that VMA is heavily influenced by the specific gravity of the aggregate (Gsb). In general, the Gsb of RAP is often inaccurate and therefore any mix including RAP could have an incorrectly measured VMA. If an asphalt mix design containing RAP is evaluated to have a VMA of 15%, but the Gsb was miscalculated, then the mix could have a lower VMA percentage, eliminating the assumed increase in asphalt binder.

Another approach to reducing gyrations is the idea of a mix design’s locking point. This is the gyration level where a mix reaches its maximum compaction and the aggregate locks together. Further compaction past the locking point could crush the aggregate within the mold. While this idea provides a valid number to cease gyrations, it does not provide a solution to the initial problem. Without an increase in VMA requirements mix designers can still adjust the gradation of a mix to minimize VMA and subsequently minimize binder content.