Evaluation of Asphalt Base Course Construction and Acceptance Requirements, Phase 1

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16. Abstract

Asphalt base is widely used in Ohio. During calendar year 2015, approximately 138,000 yd^3 (105,500 m^3) of Item 301 Asphalt Base and approximately 531,000 yd^3 (406,000 m^3) of Item 302 Asphalt Base were placed for ODOT by contract [ODOT, 2015]. Acceptance of asphalt base material, Ohio Department of Transportation (ODOT) Construction and Material Specifications (CMS) Items 301 and 302, is based on plant information, and compaction methods using specified roller types, roller weights, and coverage requirements. There is concern regarding the construction process requirements stemming from acceptance staffing reductions and some cases of premature failures. The purpose of this research was to evaluate ODOT's current acceptance methods, including an investigation of whether additional testing of the materials is warranted, whether at the plant, at the place of installation, or further specification of the mix design. Tests proposed and conducted for Phase I of this project include:

- AASHTO T 166 – Standard Method of Test for Bulk Specific Gravity (GMB) of Compacted Hot Mix Asphalt (HMA) Using Saturated Surface-Dry Specimens.
- AASHTO T 209 – Standard Method of Test for Theoretical Maximum Specific Gravity and Density of Hot-Mix Asphalt. The SSD procedure, per ODOT Supplement 1036.01 D 2, was used for cores.
- AASHTO T 283 – Standard Method of Test for Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage.
- AASHTO TP 124 – Standard Method of Test for Determining the Fracture Potential of Asphalt Mixtures Using Semicircular Bend Geometry (SCB) at Intermediate Temperature.
- TEX-245-F – Cantabro Loss.

The research team extracted and tested 720 cores from 51 project sites located in 31 counties and 11 of the 12 ODOT Districts.
Analysis of the data identified the following opportunities to improve the quality of ODOT asphalt base:

- Investigate the use of infrared imaging or nuclear density gage to reduce segregation
- Investigate the use of a density specification to reduce measured in-place air voids variability.
- Investigate the use of a thickness criteria to reduce the variability of pavement thickness.
- Investigate the use of a threshold value for Cantabro mass loss to improve durability of AC base material.
- Evaluate the impact of delivery and compaction temperature of the asphalt mixture on in-place air voids and segregation.

A plan for Phase 2 research is presented.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
	AREA			
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
	VOLUME			
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 L shall be shown in m^3.				
	MASS			
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5(°F-32)/9 or (°F-32)/1.8	Celsius temperature	°C
	ILLUMINATION			
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
	FORCE and PRESSURE or STRESS			
lbf	poundforce	4.45	newtons	N
lbf/in^2 or psi	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
	AREA			
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
	VOLUME			
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
	MASS			
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
	TEMPERATURE (exact)			
°C	Celsius temperature	1.8°C + 32	Fahrenheit temperature	°F
	ILLUMINATION			
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
	FORCE and PRESSURE or STRESS			
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2 or psi

* SI is the symbol for the International Symbol of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

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1 Executive Summary

Asphalt base is widely used in Ohio. During calendar year 2017, approximately 142,000 yd^3 (108,600 m^3) of Item 301 Asphalt Base and approximately 586,000 yd^3 (448,000 m^3) of Ohio Department of Transportation (ODOT) Construction and Material Specifications (CMS) Item 302 Asphalt Base were placed for ODOT by contract [ODOT, 2015]. Acceptance of asphalt base material, ODOT CMS Items 301 and 302, is based on plant information, and compaction methods using specified roller types, roller weights, and coverage requirements. There is concern regarding the construction process requirements stemming from acceptance staffing reductions and some cases of premature failures. The purpose of this research was to evaluate ODOT's current acceptance methods, including an investigation of whether additional testing of the materials is warranted, whether at the plant, at the place of installation, or further specification of the mix design.

A total of 728 cores were extracted from 51 project sites across the state of Ohio, representing 31 counties and 11 of the 12 ODOT Districts, which were tested to determine cracking potential, moisture susceptibility, durability, and density. Total pavement and asphalt base thickness was measured. Performance of each in service section, in terms of PCR, was obtained from ODOT. Rutting and area fatigue cracking was predicted using mechanistic/empirical models and information gathered from corresponding JMF(s), where available. Predicted truck traffic, which would have been used to design pavement thickness, was compared to the most recent measured truck traffic count. The data were used to evaluate the performance and quality of the asphalt base at each project.

Tests conducted for Phase I of this project included:

- AASHTO T 166 – Standard Method of Test for Bulk Specific Gravity (GMB) of Compacted Hot Mix Asphalt (HMA) Using Saturated Surface-Dry Specimens.
- AASHTO T 209 – Standard Method of Test for Theoretical Maximum Specific Gravity and Density of Hot-Mix Asphalt. The SSD procedure, per ODOT Supplement 1036.01 D 2, was used for cores.
- AASHTO T 283 – Standard Method of Test for Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage.
- AASHTO TP 124 – Standard Method of Test for Determining the Fracture Potential of Asphalt Mixtures Using Semicircular Bend Geometry (SCB) at Intermediate Temperature.
- TEX-245-F – Cantabro Mass Loss.

Selected findings from the research are as follows:

- Density –
  - Segregation was observed during construction or in the collected cores for seven of fifty-one projects.
  - The minimum temperature of the mix at the paver specified by ODOT, 250°F (121°C), is on the low end of temperatures specified by other states reviewed.
- Air voids –
  - Measured in-place air voids of the top AC base lift were highly variable. Values among all 714 core samples ranged from 1.1% to 14.4%, with average and standard deviation of 5.91% and 2.19%, respectively. AC bases from project sites with poor or average field performance tended to have greater spread in in-place air voids.

    - Four of the five project sites with exceptional performance had AC base with average in-place air voids between 4.0% and 6.5%, while the fifth had average in place air voids of 8.0%.
- Thickness – Current practices do not adequately control pavement thickness. Although core height closely matched planned thickness on average, approximately 20% of cores were at least 1 in (25 mm) deficient.
- Density –
    - As expected, the tensile strength of the conditioned specimen and TSR decreased with increasing average in-place air voids, although the relationships were weak. The majority of project sites with poor field performance had TSR values for AC base mixes less than the 0.70 criterion.
    - A moderate relationship was found between Cantabro mass loss and TSR, indicating an increase in mass loss is related to a decrease in TSR. Furthermore, it was found AC base mixes with mass loss greater than 21% tended to have TSR values less than the established criterion of 0.70.
    - It is well known asphalt binder content plays an integral role in fatigue resistance and mix durability, however due to multiple JMFs approved on a project, the effect of binder content on laboratory testing, field performance, and in-place air voids could not be evaluated in this study.

A second phase of this project is warranted to investigate the following:

- Segregation was seen in asphalt mat and cores. Methods to identify segregation during paving should be investigated.
- Measured in-place air voids were highly variable. There is a need to determine if a density specification is required.
- Current practices do not adequately control pavement thickness. An investigation of other methods to ensure as-built thickness is adequate is needed.
- A threshold value for Cantabro mass loss for ODOT AC base material should be investigated further.
- Based on the review of specifications from other states, it is recommended delivery and compaction temperature of the asphalt mixture be monitored to evaluate the impact on in-place air voids and segregation.

2 Project Background

Acceptance of asphalt concrete base, Ohio Department of Transportation (ODOT) Construction and Material Specifications (CMS) Items 301 and 302, is based on plant information, and compaction methods using specified roller types, roller weights, and coverage requirements. There is concern regarding the construction process requirements stemming from acceptance staffing reductions and some cases of premature failures.

Flexible pavements use multiple layers of various materials to carry traffic by distributing the load over an increasingly larger area as the depth increases. This allows the use of lower cost, less stiff material with depth. The buildup of a new flexible pavement constructed by ODOT consists of dense graded aggregate base, asphalt base, asphalt intermediate course, and an asphalt surface course. The asphalt base layer is typically the largest contributor to the pavement structural number. The surface and intermediate courses must be designed to resist rutting, cracking, and oxidation to perform well. The asphalt base, being located deeper in the pavement structure, must be designed to resist moisture and cracking, rutting being less of a concern due to low compressive stress at this depth in the pavement.

Asphalt base is widely used in Ohio. During calendar year 2017, approximately 142,000 yd^3 (108,600 m^3) of Item 301 Asphalt Concrete Base and approximately 586,000 yd^3 (448,000 m^3) of Item 302 Asphalt Concrete Base were placed for ODOT by contract for a total cost of \$69.3 million [ODOT, 2015]. The cost of a base failure is high. Depending on the extent of failure on a project; full depth repair, a thick structural overlay, or reconstruction may be required to correct the failure. The purpose of this research was to evaluate ODOT's current acceptance methods, including an investigation of whether additional testing of the materials is warranted, whether at the plant, at the place of installation, or in further specification of the mix design.

3 Research Context

To address a reduction in workforce, ODOT, like many state DOTs, is evolving from materials and methods specifications to quality control/quality assurance (QC/QA) procedures. These specifications shift responsibility, as well as risk, to the contractor while allowing for innovation and reducing the need for agency inspectors. For ODOT CMS Item 301, ODOT's current specifications require the contractor submit material samples to the ODOT central laboratory where the required binder content is determined. For ODOT CMS Item 302, the contractor designs a mix which is submitted to the ODOT central laboratory for approval. For ODOT CMS Item 302, Tensile Strength Ratio testing is also required to determine if antistrip additives will be needed. The approved job mix formula (JMF) is then used by the plant to produce the mix delivered to the worksite for placement. For both Items, material is sampled during production to verify binder content and aggregate gradation (% passing the #4 sieve) are within tolerances. Thickness is monitored by comparing tonnage placed per station to a required placing rate based on volume of material shown on the plans and an ODOT laboratory provided conversion factor.

Item 301 was the sole hot mix asphalt base available to the ODOT designer for decades. However, by the 1990s, the Interstate system had aged to the point where major rehabilitation or reconstruction was necessary for many of the rigid pavement sections. At the same time, ODOT had adopted a mechanistic based overlay design process which, when combined with the increasing truck traffic, resulted in thick overlays. As a result, asphalt base was incorporated into overlay buildups more often. To increase stability and reduce cost of asphalt base, a mix utilizing a large stone aggregate gradation was investigated. Abdulshafi et al [1992] concluded a large aggregate base could be used in lieu of Item 301 Bituminous Aggregate Base. They also concluded the gradation curves and limits on uniformity and curvature coefficients are important factors, unconfined compressive strengths and resilient modulus values are higher than the Item 301 mixes, and rutting resistance is superior to Item 301. In 1992, ODOT introduced a plan note modification to Item 301, with a coarser gradation. This mix, with modification to reduce segregation and improve durability, was included as Item 302 in the 1995 CMS. Design air content was reduced from 5% to 4.5% with the 2002 specifications and to 4% with the 2008 specifications. Supplemental specification 880 (2005) permitted the use of up to 30% reclaimed asphalt pavement (RAP) provided a virgin binder content of no less than 3.4% was used.

A national effort has been ongoing to improve asphalt mix design and testing through the National Cooperative Highway Research Program (NCHRP). NCHRP Project 9-33 developed improved mix design procedures for dense-graded, open-graded and gap graded hot mix asphalt as well as mixes containing RAP. A mix design manual, NCHRP Report 673, incorporating advancements since the conclusion of the Strategic Highway Research Program (SHRP) Superpave project, and an Excel spreadsheet tool was developed [Advanced Asphalt Technologies, 2011]. NCHRP Project 9-39 identified improved procedures for determining mixing and compaction temperatures for hot mix asphalt lab test specimens incorporating performance grade (PG) asphalt binder. The results are presented in NCHRP Report 648 [West, et al., 2010]. NCHRP Project 9-46 addressed methods to prepare and characterize mixes with high RAP content. This information was published as NCHRP report 752 [West, Willis, and Marastreanu, 2013].

There are many guides for mix design, plant operation, lay down, compaction, and quality control of asphalt bases. One of the widely referenced guides is the *Hot Mix Paving Handbook* developed by the Transportation Research Board and recommended by Federal Aviation Administration Advisory Circular AC 150/5370-14B [FAA and USACE, 2013].

Asphalt base with a density requirement has been used on an experimental basis in Ohio. Other non-destructive testing methods for monitoring density, segregation, etc., have been investigated including the falling weight deflectometer (FWD), ground penetrating radar (GPR), portable seismic pavement analyzer (PSPA), intelligent compaction, and thermal imaging, with promising results. [Hanna, 2002; Sargand et al, 2009]

The goal of this project is an improvement of ODOT's asphalt base acceptance methods, which is anticipated to result in an improvement in the quality of asphalt base construction in pavement structures. Specific objectives include:

1. Review mix design and acceptance procedures used in other states for asphalt base courses to identify best practices and opportunities to improve ODOT methods.
2. Evaluate the adequacy of ODOT's current practice, in particular whether it leads to repeatable and uniform results in asphalt bases.
3. Suggest modifications to ODOT's acceptance methods for asphalt bases.

To achieve these objectives, the following tasks were undertaken:

1. Create a synthesis of current practice. Construction and materials specifications of other state DOTs were reviewed and compared and contrasted with ODOT's specifications for Items 301 and 302.

2. Devise test plan. A materials sampling and testing program was developed to evaluate the low temperature cracking potential, moisture susceptibility, compaction effort, and durability of asphalt base mixtures.
3. Obtain samples of asphalt base materials. Various records from ODOT were reviewed to identify potential sample sites. Forty seven projects were selected, some with both Item 301 and 302 included on the project. 630 cores were collected for testing.
4. Measure specimens in the laboratory. Theoretical Maximum Specific Gravity (Gmm), Bulk Specific Gravity (Gmb), Semicircular bend (SCB), Tensile Strength Ratio (TSR), and Cantabro tests were performed on the collected samples.
5. Review construction records. Available JMF records were reviewed. JMF data was used to evaluate rutting and fatigue potential of the mix.
6. Develop recommendations for advanced testing in Phase 2. A test plan is included in this report.
7. Prepare interim report.

4 Synthesis of current practice: Specification Review

Specifications of asphalt bases throughout the country were studied to compare and contrast with Ohio DOT specifications. After initial review it was found that many state agencies have not established unique specifications specifically for asphalt base courses and adopt similar specifications for all asphalt layers (Surface, binder/leveling, base). The specification review summary consists of two comparisons. The first comparison focuses on a broad overview of the requirements for each agency that had a specific specification for asphalt concrete (AC) base courses and other agencies who did not have specific AC base specifications. The second comparison summarizes specific values for the different specifications and how they compare to Ohio DOT specifications.

The specification search included several state transportation agencies surrounding Ohio and across the US and Canada. The six state agencies with specifications for AC base courses are listed below:

- Michigan
- Indiana
- Pennsylvania
- South Carolina
- Virginia
- Texas

The eight other states and one Canadian providence which did not have specifically designated AC base specifications but did have specifications for 19 mm (0.75 in) and 25 mm (1 in) mixtures include:

- Kentucky
- Illinois
- Ontario
- Wisconsin
- Maryland
- California
- Colorado
- Kansas

Several other states were looked at but no detailed specifications were obtained or the specifications did not have a clear representation of their mixture, aggregate, or acceptance criteria. The bar charts in Figure 4-1 through Figure 4-3 summarize the results from the broad agency specification review. It can be seen in Figure 4-1 the majority of the fourteen agencies had AC mixture related specifications including requirements regarding gradation, binder content, binder type/grade, RAP content, and VMA. Figure 4-2 summarizes the results for specifications specifically related to the aggregates used in the AC mixtures. A greater variation is observed for the aggregate specifications from agency to agency, with none adopted by more than five agencies, and a number for one agency only. The most prevalent specifications regarded Los Angeles Abrasion Test, angularity, sulfate soundness, and percent sand. An even larger variation was observed for the types of acceptance testing performed on AC base courses, as summarized in Figure 4-3. The most common acceptance tests were density, binder content, air voids, aggregate gradation, VMA, and VFA. It should be noted that these summaries do not include ODOT specifications.

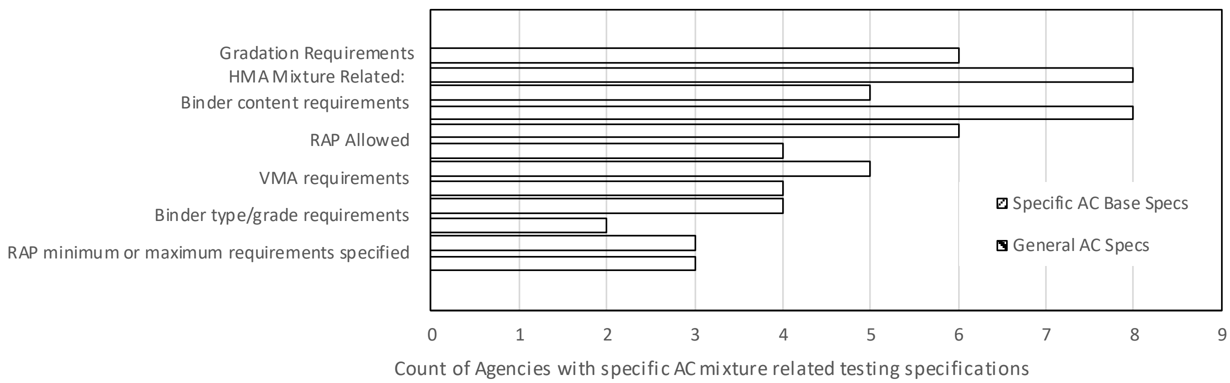

Figure 4-1. Summary of state agencies with AC mixture related specifications

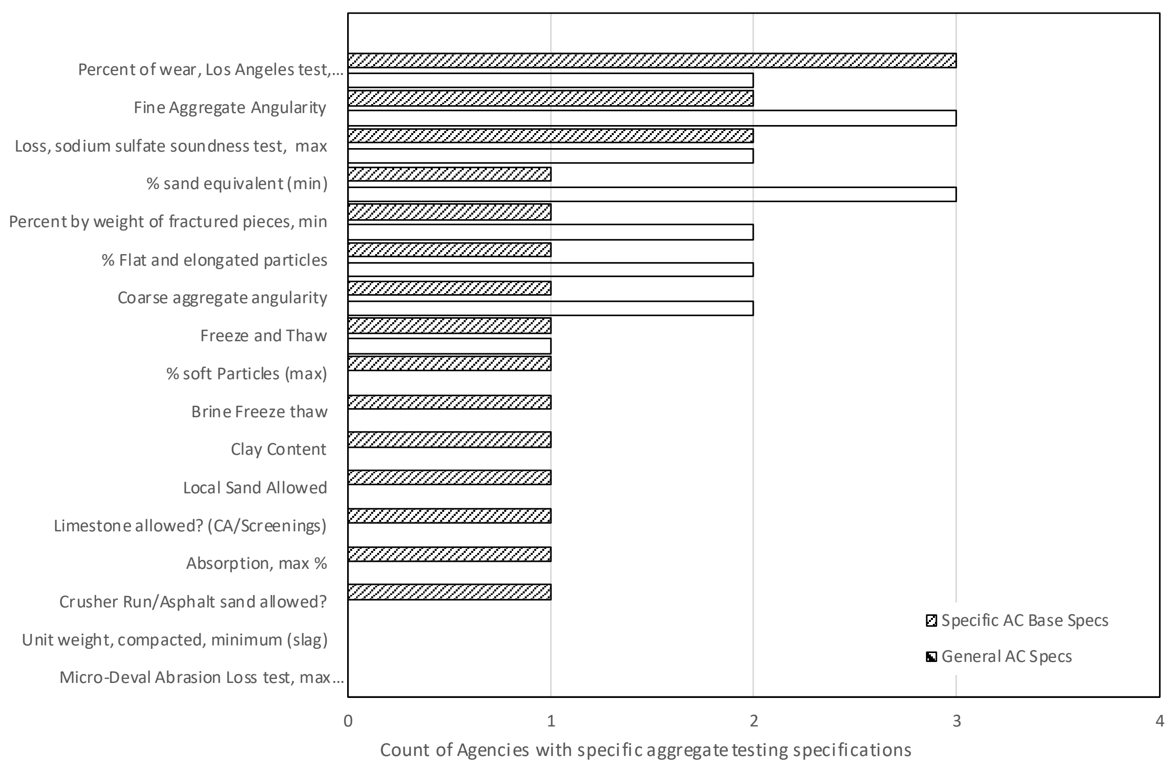

Figure 4-2. Summary of state agencies with specific aggregate specifications

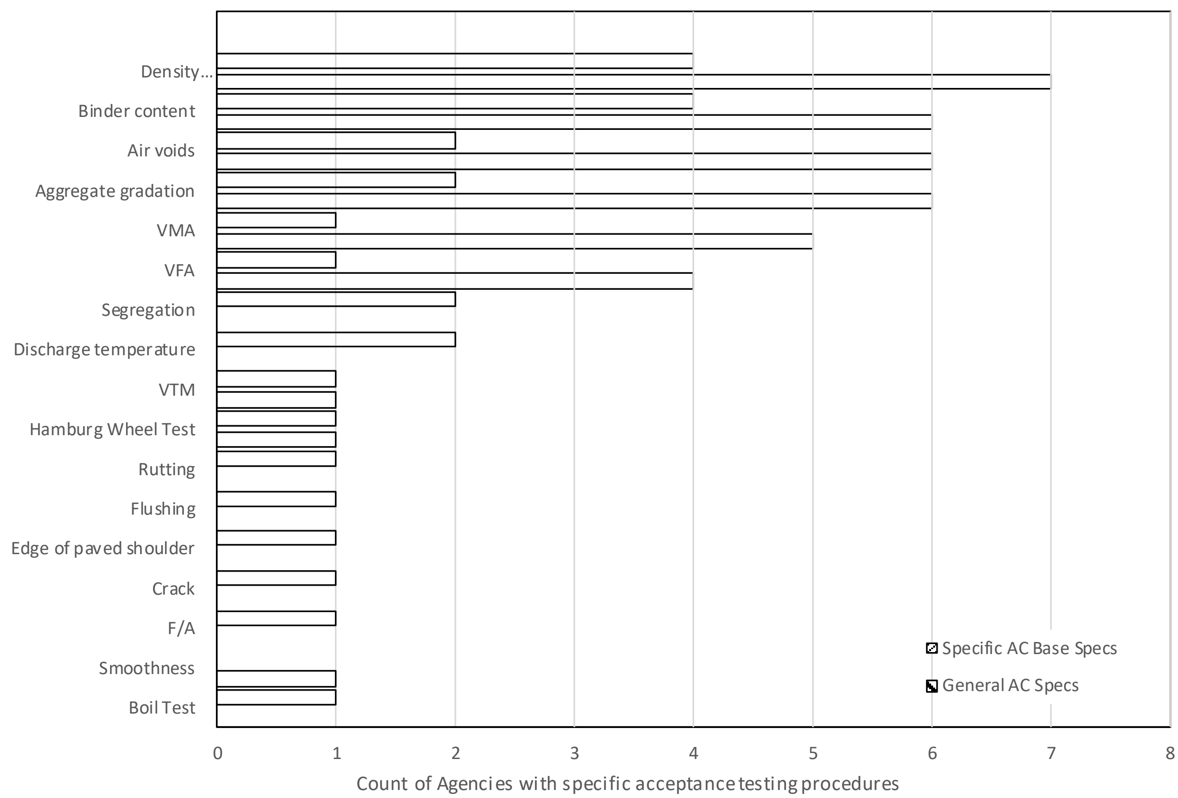

Figure 4-3. Summary of state agencies with acceptance and testing related specifications

4.1 Summary of Ohio Specifications 301 and 302

A brief summary of the two different Ohio asphalt base specifications are discussed.

4.1.1 Gradation

The grain size distribution limits for ODOT Items 301 and 302 differ slightly in relation to the minimum and maximum values. Figure 4-4 compares the two specifications. Ohio 302 specification has almost similar minimum values. The bigger difference is between the maximum requirement for each sieve where the 302 specification has much lower maximum limits for each sieve.

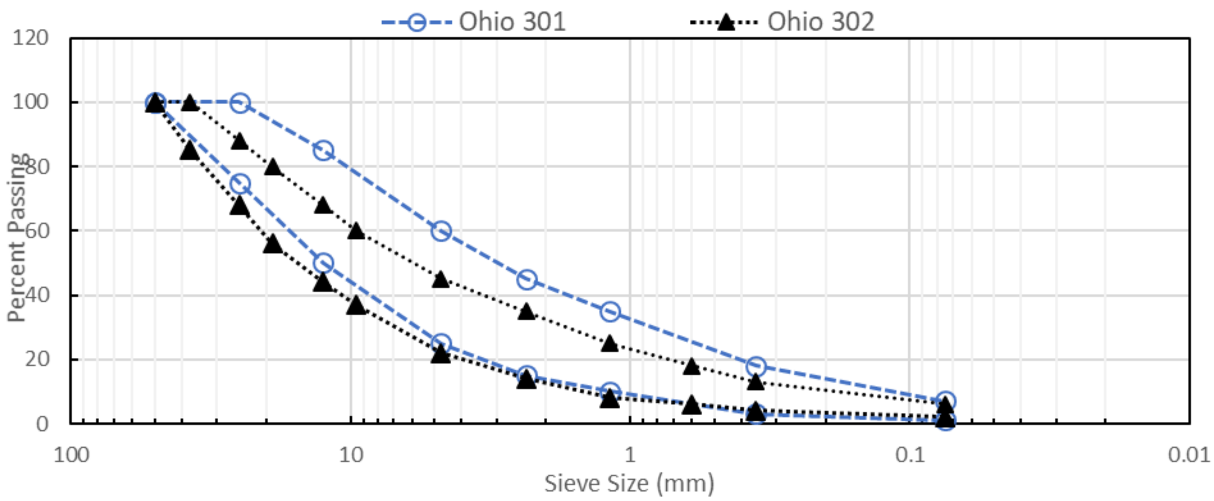

Figure 4-4. Grain size distribution limits for Ohio Item 301 and 302

4.1.2 Mixture and Physical properties

The mixture properties for both specifications are summarized in Table 4-1. The mixture aggregate physical property specifications are summarized in Table 4-2. The aggregate properties do not differ between the two specifications.

Table 4-1. Summary of mixture properties for Ohio Item 301 and 302 specifications

Mixture Property	**Ohio Item 301**	**Ohio Item 302**
Binder type and grade	PG 64-22	
Binder content	4.7 to 7 %	6 % max
VMA	-	12 min
RAP	Yes (spec 401.04)	-
Percent RAP by Dry weight of mix (max)	50	40
RAS usage	Manufacturing waste and Tear-offs	Manufacturing waste and Tear-offs
Total Virgin Asphalt binder content (min)	2.7	2
Minimum depth	3 inches	4 inches
Maximum depth	6 inches/lift	7.75 inches (greater requires additional lifts)
Minimum Temperature	250°F	250°F
Spreading and Surface Tolerances	<3/8 inch in surface variation from testing edge of a 10-foot straightedge	<3/8 inch in surface variation from testing edge of a 10-foot straightedge
Payment	Cubic yard/Cubic meter	Cubic yard/Cubic meter

Table 4-2. Summary of mixture aggregate physical properties for Ohio Item 301 and 302 specifications

Percent of wear, Los Angeles test, maximum	50%
Unit weight, compacted, minimum (slag)	65 lf/ft^3
Loss, sodium sulfate soundness test, max	15%
Percent by weight of fractured pieces, min	40%
Micro-Deval Abrasion Loss test, max (coarse aggregate gravel only)	22%

4.1.3 Acceptance

The quality control acceptance requirements for ODOT Item 301 and 302 are summarized in Table 4-3.

Table 4-3. Summary of quality control limits for ODOT Item 301 and 302 specifications

Quality control item	Item 301	Item 302
Binder content acceptance	± 0.5%	± 0.5%
No.4 Sieve acceptance	± 6%	± 7%
Method	AASHTO T248	-

4.2 Mixture Design and Acceptance Values for Identified Agencies

Detailed comparisons between ODOT AC base specifications and those of other state agencies are divided into several categories. These categories are:

- AC Base Mixture information
- Aggregate physical properties
- Quality control and acceptance

4.2.1 AC Base Mixture information

The AC base mixture information was obtained from specifications and asphalt material design guides available for each state agency. The mixture properties consist of the aggregate gradation, binder type/grade, binder content, air voids, inclusion of recycled asphalt pavement (RAP)/recycled asphalt shingles (RAS), asphalt temperatures, and spreading/surface tolerances. Not all of these available for every state agency.

- **Gradation:** The AC mixture gradations for the majority of the agencies were similar to ODOT specs. Aggregate mixtures ranged from 19mm to 37.5mm. Most agencies had specifications for 19 mm (0.75 in) and 25 mm (1 in) mixtures. South Carolina had four different base mixes including two coarse graded and two fine graded mixtures.

- **Binder type/grade:** Binder type/grades varied from agency to agency. The ODOT Pavement Design Manual mentioned the use of a PG64-22 binder for the base courses. The ODOT CMS provides guidelines for the use of PG58-28 or PG64-28 binder when RAP is incorporated into the mix. Michigan DOT does not have a specific binder type for AC bases and adopts the same methodology for all binder types. The majority of the agencies in this comparison specified a PG64-22 binder which is identical to ODOT.

Additionally, Virginia specifies only a minimum binder grade. Pennsylvania specifies either a PG58-28 or a PG64-22.

- **Binder content:** The binder content for all comparisons ranged from 4% to 6% which is slightly lower than ODOT specifications of 4.7% to 7% for ODOT Item 301 and 6% maximum for ODOT Item 302.
- **Air voids:** The majority of agencies did not have specific mix design air void criteria for the AC base courses.
- **Other properties:** Other properties for base mixture design including VMA, VFA, and Fines to effective binder ratio were identified. The ODOT Item 302 mixture has a specification of 12% for VMA which is similar to all other agency specifications. The other agency specifications ranged from 11% to 13%. The VMA values for other agencies varied between 65% and 87% and is dependent on traffic. The fines ratio for other agencies ranged from 0.6 to 1.6. ODOT did not specify limits for the fines ratio in their specification.
- **Recycled materials (RAP, RAS):** The ODOT specifications indicate recycled materials are allowed for the AC base layers. The ODOT Item 301 and Item 302 specifies a maximum RAP percent of 50% and 40%, respectively, can be included in the mixture. ODOT also allows up to 5% RAS in all AC mixtures. Other agencies allow the use of RAP/RAS in their mixtures. Not all specified the amount however, Texas and Indiana specified a maximum of 40% for their mixtures.
- **Asphalt Mixture Temperatures:** The minimum delivery temperature for ODOT Item 301 and ODOT Item 302 is 250°F (121°C). Other agencies provide a minimum and maximum temperature as well as the compaction temperature for their asphalt base course mixtures. Pennsylvania also specified their mixing temperatures based on the binder grade. The range of minimum mixing temperatures were from 250°F (121°C) to 285°F (141°C) and maximum temperature from 290°F (143°C) to 330°F (166°C). Virginia also specified the compaction temperature for their asphalt base mixtures which ranged from 295°F (146°C) to 320°F (160°C).

4.2.2 Acceptance Values

Acceptance values for AC Bases included lift thickness and spreading and surface tolerances.

- **Lift thicknesses:** The ODOT Item 301 and ODOT Item 302 mixes indicate a minimum lift thickness of 3 and 4 inches should be used and a maximum of 6 in (152 mm) and 7.75 in (197 mm), respectively. For ODOT Item 302, the base materials may be placed in two lifts if the plan thickness is between 7 and 7.75 inches and 95% percent passing the 1.50 in (37.5 mm) sieve has been confirmed by the state. These values are in conjunction with other agencies where the lift thicknesses ranged from 3 in (76 mm) to 6 in (152 mm).
- **Spreading/surface tolerances:** The ODOT specifications indicate the surface variation from the testing edge of a 10 ft (3.0 m) straightedge should be less than 3/8 in (9.5 mm) for both ODOT Item 301 and ODOT Item 302. Michigan and Pennsylvania also included surface tolerances in their specifications. Michigan specifies that a 3/4 in (19 mm) tolerance for lower AC base layer is acceptable and 3/8 in (9.5 mm) for the final layers. Pennsylvania requires a 1/4 in (4.2 mm) tolerance in their specifications.

4.2.3 Aggregate Physical Properties

The aggregate physical properties are specified for many of the agencies listed above. The various properties are discussed next.

- **Percent of wear, Los Angeles Abrasion Test:** Ohio specifications indicate a maximum value of 50% is allowed for AC base layers. Several other agencies also specified the LA Abrasion test with values ranging from 35% to 60%.
- **Unit weight, compacted, minimum (slag):** ODOT specifies a value of 65 lb/ft^3 (1040 kg/m^3). Only Indiana had a specific specification for this parameter with a value of 70 lb/ft^3 (1120 kg/m^3).
- **Loss, sodium sulfate soundness test:** The specified value for the percent loss is 15% according to ODOT specifications. Indiana specified 10% and 16% for fine and coarse mixtures respectively. South Carolina specifically indicates there are no requirements.
- **Percent by weight of fractured pieces:** ODOT was the only state which directly specified the percent by weight of fractured pieces. Virginia does specify a minimum of 80% of the mixture must have 1 fractured face and 75% must have 2 fractured faces.
- **Brine Freeze Thaw:** Indiana requires a Brine Freeze Thaw test for their aggregates. No other agency requires this test, including ODOT.
- **Micro-Deval Abrasion loss test:** ODOT requires the Micro-Deval Abrasion loss test for coarse aggregate gravel only. A value of 22% is specified. Only Texas indicated they use the test and did not specify any values. The test is also only performed by the Texas Engineer and not required by contractors.
- **Aggregate Angularity:** Michigan, Indiana and Virginia specify a value for fine aggregate angularity. The values range from 40% to 45%. Only Indiana specified a value for coarse aggregate angularity with a value ranging from 50% to 100% depending on traffic. ODOT did not have a specific value for AC Base mixtures.
- **Other Aggregate Properties:** Michigan and Virginia specified values for percent sand equivalent and percent flat and elongated particles. The values ranged from 40% to 50% for percent sand equivalent and 10% maximum for flat and elongated particles.

4.3 Quality Control and Acceptance Specifications.

The quality control and acceptance specifications vary between all the different agencies. There were few consistencies between the various agencies. Therefore, this section discusses the various agencies individually instead of comparing the specifications to those of ODOT.

4.3.1 Binder Acceptance

The majority of agencies have binder content acceptance criteria. ODOT specifies the binder content must be within ± 0.5% of the job mix formula values. Other agencies have similar criteria and range from ± 0.5% to 0.7%. Texas indicates the binder content values need to be within 0.3% between the contractor and the TxDOT engineer.

4.3.2 Density Testing

Some agencies have specific density testing performed for their AC base courses. Many specified only the number of tests to perform per lot or sublot. Others indicated acceptance is based on a percentage of target density. Once the tests have been performed the agency will observe and document the density readings equal to or greater than 10% of the tests. Texas specifications indicate a maximum allowable density range from highest to lowest values of 8.0 lb/ft^3 (128 kg/m^3) and a maximum allowable density range between the average and lowest value of 5.0 lb/ft^3 (80 kg/m^3). Pennsylvania requires asphalt mat density for base courses be between 0.89 and 1.00 times the maximum theoretical density.

4.3.3 Segregation

Michigan and Texas have specifications for segregation. Michigan requires replacing the AC pavement if the segregation exceeds 215 ft^2 (30 m^2) or 328 ft (100 m) lane length. Texas has a different approach and requires continuous thermal imaging or temperature monitoring of the asphalt mat during construction of all asphalt layers. The thermal imaging can be performed using an infrared scanner/bar mounted to the paver which spans the entire width of paving, or an infrared camera. Specification Tex-244-F, Thermal Profile of Hot Mix Asphalt, details the criteria to determine potential locations for segregation in the uncompacted mat of hot mix asphalt. The temperature differential between the thermal profile sensor locations are determined for each 150 ft (46 m) segment of paving. The segregation criteria are as follows:

- 0°F (-18°C) to 25°F (-3.9°C) – Minimal thermal segregation
- 25°F (-3.9°C) to 50°F (10°C) – Moderate thermal segregation
- >50°F (10°C) – Severe thermal segregation

The contractor must send the collected temperature data to the State Agency for review and calculate the thermal segregation potential based on the temperature differential calculations. Based on the results, the potential segregation areas can be identified which may exhibit density issues after compaction.

4.3.4 Other Criteria

Other acceptance testing include testing for rutting, flushing, and the edge of the paved shoulder. The corrective actions for not meeting the criteria are to replace the section or to trim the edges. Pennsylvania also includes a weather restriction that does not allow for base course construction on prepared surfaces which are wet or when the surface temperature is below 35°F (1.7°C). Pennsylvania also requires a full depth core for each 3000 yd^2 (2500 m^2) of base course placement for thickness measurements. The thickness tolerance should be within ½ in (13 mm). Texas has an extensive process for reporting during construction procedures. All reports must be reported within one working day after sublot completion, following the process below:

Production quality control: Reported by contractor sent to state engineer

- Gradation tests
- Asphalt binder content
- Laboratory molded density
- Moisture content
- Boil test

Production quality assurance: Reported by state engineer sent to contractor

- Gradation tests
- Asphalt binder content
- Laboratory molded density
- Hamburg wheel test – optional

- Boil test – frequency specified on plans
- Binder tests – optional

Placement quality control: Reported by contractor sent to state engineer

- In-place air voids
- Segregation
- Longitudinal joint density
- Thermal profile

Placement quality assurance: Reported by state engineer sent to contractor

- In-place air voids
- Segregation
- Longitudinal joint density
- Thermal profile
- Aging ratio

5 Research Approach

5.1 Site selection

To gain an understanding of the effects of specification changes, use of RAP/RAS, and geographic location, and also to evaluate the causes of failure, the experimental matrix in Table 5–1 was developed. It would be desirable fill all cells in the matrix with replicate projects for a total of 160 sites. However, due to limited time, funds, and availability of projects, the research team proposed a total of 50 sites selected to obtain a geographic spread of projects representing five regions in the state (northwest, northeast, southwest, southeast, and central) and complete the work within a reasonable time.

To identify candidate projects, four sources of information were utilized:

- A survey was distributed to ODOT Districts. The intent of the survey was to identify active projects and projects which have failed prematurely. Eight of twelve ODOT Districts responded (see Appendix B for survey and response summary).
- A query of the ODOT Hummingbird database was performed to identify projects with more than 1173 yd^3 (897 m^3) of ODOT CMS Item 301, 302 or 880. 1173 yd^3 (897 m^3) is the volume of material in one 12 ft (3.66 m) wide lane mile (1.6 km) of 6 in (152 mm) thick pavement.
- Using ODOT's pavement management data, the Office of Pavement Engineering provided a list of pavements known to have an asphalt base.
- The list of projects approved by the pavement selection committee was obtained from the ODOT Office of Pavement Engineering.

The availability of projects limited the ability to obtain a geographic spread of projects. A majority of new construction was located in the Toledo and Columbus area so most of the control sections were located in northwest and central Ohio. The fewest number of past projects and control projects were located in southeast Ohio.

Table 5–1. Experimental matrix used in this study, northern Ohio sections.

Region		NW		NE	
Specification		301	302	301	302
1995 thru 2002 specs design air = 5.0%	C-R-S	LUC-2-21.24	FUL-20-10.86	WAY-30-11.86 EB	CUY-71-0.00
	Project No	990141	000341	040044	980748
	PID	9159	19342	16285/16287	15717
	C-R-S	MER-219-14.04	MER-219-14.04		MED-71-9.56
	Project No	050313	050313		050343
	PID	19968	19968		14018
	C-R-S				
	Project No				
	PID				
	C-R-S				
	Project No				
	PID				
2005+ specs, 880 with RAP up to 30% RAP if virgin binder > 3.4%. Design air = 4.5% '02 to '08	C-R-S		HAN-30-3.00		SUM-77-17.20
	Project No		050003		060151
	PID		77302		16514
	C-R-S		DEF-24-7.96		
	Project No		060087		
	PID		24337		
2008+ specs design air = 4.0%	C-R-S	WOO-75-19.43			LAK-2-7.60
	Project No	140237			100215
	PID	25521			79545
	C-R-S				
	Project No				
	PID				
	C-R-S				
	Project No				
	PID				
Poor Performance	C-R-S		WOO-795-2.01		WAY-30-11.86 WB
	Project No		990505		040044
	PID		13725		16285/16287
	C-R-S				ASD-39-0.00
	Project No				040500
	PID				23578
	C-R-S				
	Project No				
	PID				
	C-R-S				
	Project No				
	PID				
Control	C-R-S		WOO-75-10.61	MED-42-17.68	CUY/SUM-77-0.00/32.73
	Project No		140170	160430	163019
	PID		95435	92954	79671
	C-R-S		WOO-75-2.37N		
	Project No		140199		
	PID		95436		
	C-R-S		HAN/WOO-75-19.22/0.00		
	Project No		143000		
	PID		95437		
	C-R-S		LUC-75-6.70		
	Project No		140536		
	PID		76032		

Table 5–2. Experimental matrix used in this study, central Ohio sections.

Region		Central		
Specification		301	302	880
1995 thru 2002 specs design air = 5.0%	C-R-S Project No PID	GRE/FAY-35-26.20/0.00 000091 4388	COS-36-20.83 960278 14142	
	C-R-S Project No PID	MAD-142-0.49 010317 11739	DEL-23-19.24 970335 16350	
	C-R-S Project No PID	FAI-33-17.44, service road 13 02446 16295		
	C-R-S Project No PID	FAI-33-19.79, service road 18 030046 23057		
2005+ specs, 880 with RAP up to 30% RAP if virgin binder > 3.4%. Design air = 4.5% '02 to '08	C-R-S Project No PID			FRA/LIC-161-23.20/0.00 060150 24486
	C-R-S Project No PID			
2008+ specs design air = 4.0%	C-R-S Project No PID		DEL-23-17.64 120284 79370	
	C-R-S Project No PID		MRW-71-3.17 133001 86920	
	C-R-S Project No PID			
Poor Performance	C-R-S Project No PID		FAY-35-2.57 000577 9078	FAI-33-7.31 010136 16293
	C-R-S Project No PID			FAI-33-13.25 20110 16294
	C-R-S Project No PID			FAI-33-17.44 02446 16295
	C-R-S Project No PID			FAI-33-19.79 030046 23057
Control	C-R-S Project No PID	LIC-161-1.83 168030 97879	FRA-270-21.67 150249 81747	
	C-R-S Project No PID		FRA-71-5.29 150395 84868	
	C-R-S Project No PID		FRA-70-3.41 150396 25594	
	C-R-S Project No PID		FRA-270-35.41 170003 84620	

Table 5–3. Experimental matrix used in this study, southern Ohio sections.

Region		SW			SE	
Specification		301	302	880	301	302
1995 thru 2002 specs design air = 5.0%	C-R-S Project No PID	HAM-264-6.87 000135 13853	WAR-71-3.78 990780 10696	ROS-207-0.00 040533 18492		
	C-R-S Project No PID					
	C-R-S Project No PID					
	C-R-S Project No PID					
2005+ specs, 880 with RAP up to 30% RAP if virgin binder > 3.4%. Design air = 4.5% '02 to '08	C-R-S Project No PID			CLI-73-12.03 060413 78569	GAL-35-8.32 070334 22520	
	C-R-S Project No PID					
2008+ specs design air = 4.0%	C-R-S Project No PID		BUT-75-5.91 080246 75971	CLI-73-6.52 090244 78571		
	C-R-S Project No PID		CLA-70-13.98 100243 84664			
	C-R-S Project No PID		WAR-71-14.20 100280 22950			
Poor Performance	C-R-S Project No PID				VIN-50-11.75 010123 10504	
	C-R-S Project No PID					
	C-R-S Project No PID					
	C-R-S Project No PID					
Control	C-R-S Project No PID					ATH-33-11.74 170008 84468
	C-R-S Project No PID					
	C-R-S Project No PID					
	C-R-S Project No PID					

5.2 Field work

Based on the literature review in Appendix A, the sampling and testing plan, as described in Appendix D, was developed to investigate the potential for low temperature cracking, moisture susceptibility, mix durability, and in-place density.

A minimum of fourteen 6 in (152 mm) diameter cores per project were needed for the proposed laboratory testing, which is described in the next section of this report. A 1400 ft (427 m) length of road was arbitrarily chosen to represent a lot. Each section was divided into 7 sublots. Coring locations were randomly selected following ODOT Test Procedure TE-217. Field sheets, such as shown in Table 5–4 for FRA-71-5.29, were developed for each test section. Once traffic control, provided by ODOT, was in place, the locations were marked and cores removed. The majority of the coring was completed by Ohio University. ODOT assisted by coring sections in central Ohio to reduce time needed to obtain samples.

Table 5–4. Randomized coring locations generated for Project FRA-71-5.29.

County-Route-Section-Direction			FRA-71-5.29-N						4/27/2017	
Project Number			150395							
Item Number			302							
JMF			B170145							
RAP?			Yes							
Beginning Station			280+00							
Ending Station			294+00							
Total Length (ft)			1400							
Number of Lots			7							
Lane Width (in)			144							
Sublot Length (ft)			200							
Sublot	Core #	Longitudinal (Station)				Transverse (in)				Remarks/Notes
		Beg. Sta.	End Sta.	Length (sta)	Random Number	Lower	Upper	Length	Random Number	
1	1	280+00	282+00	200	281+29	6	132	126	107	Top Lift + Bottom Lift = (7"+5.75")=12.75"
	2	280+00	282+00		281+84	6	132	126	24	(6.75"+6")=12.75"
2	3	282+00	284+00	200	282+11	6	132	126	24	(4.5"+6.25")=10.75"
	4	282+00	284+00		284+00	6	132	126	109	(6.5"+6")=12.5"
3	5	284+00	286+00	200	284+58	6	132	126	132	(5.25"+6.5")=11.75"
	6	284+00	286+00		285+44	6	132	126	122	(5.75"+6.5")=12.25
4	7	286+00	288+00	200	287+79	6	132	126	8	(5.75"+6.25")=12"
	8	286+00	288+00		287+98	6	132	126	76	(5.5"+6.5")=12"
5	9	288+00	290+00	200	289+66	6	132	126	85	(5.5"+6.25")=11.75"
	10	288+00	290+00		289+72	6	132	126	92	(5.5"+6.25")=11.75"
6	11	290+00	292+00	200	291+00	6	132	126	29	(6.5"+5.5")=12"
	12	290+00	292+00		291+02	6	132	126	79	(6.5"+6")=12.5"
7	13	292+00	294+00	200	292+98	6	132	126	20	(6"+6")=12"
	14	292+00	294+00		293+43	6	132	126	15	(6.75"+5.25")=12"

5.2.1 Asphalt base layer thickness determination from cores

The sampling and testing plan was focused on material testing to estimate performance and did not include the measurement of thickness. However, the cores obtained from MAD-142 showed a high variability in thickness. As a result, an evaluation of thickness variability was undertaken. Asphalt base and total pavement thickness was measured in the field or estimated from cores in the laboratory.

5.3 Laboratory Testing

Five tests were identified in the sampling and testing plan for the Phase 1 laboratory investigation:

1. AASHTO T 166 – Standard Method Of Test For Bulk Specific Gravity Of Compacted Hot Mix Asphalt (HMA) Using Saturated Surface-Dry Specimens. Cores were not cured.

2. AASHTO T 209 – Standard Method Of Test For Theoretical Maximum Specific Gravity And Density Of Hot-Mix Asphalt. Samples were not cured at elevated temperatures, and tests were not performed while samples were warm. The SSD procedure, per ODOT Supplement 1036.01 D 2, was used for cores.

3. AASHTO T 283 – Standard Method Of Test For Resistance Of Compacted Asphalt Mixtures To Moisture-Induced Damage

4. AASHTO TP 124 – Standard Method Of Test For Determining The Fracture Potential Of Asphalt Mixtures Using Semicircular Bend Geometry (SCB) At Intermediate Temperature

5. TEX-245-F – Cantabro Loss Test

The Laboratory testing procedures are discussed in detail in Appendix H, with results discussed in Appendix I through Appendix L.

6 Fatigue and Rutting Performance Prediction

A mixture evaluation procedure was used to predict a dense-graded asphalt concrete (AC) mixture's resistance to area fatigue cracking and rutting in accordance with the Mechanistic-Empirical Pavement Design Guide (MEPDG) software. The evaluation procedure estimates the fracture and permanent deformation constants from mixture volumetric parameters, similar to the regression equations used to calculate dynamic modulus, and was formalized under NCHRP 1-40B when fundamental fracture fatigue strength and repeated load plastic deformation tests were unavailable.

The procedure outlined in Appendix M were used to estimate the fatigue fracture coefficients for each AC base mixture included in the study. Total rut depth and bottom-up fatigue cracking were predicted using a constant structure and site condition factors so the only difference in the predictions was the AC base mixtures.

Three categories of performance or fatigue cracking were used to rank the AC base mixtures, as listed below.

- Crack resistant, less than 25 percent predicted fatigue cracks over 20 years.
- Average, 25 to 40 percent fatigue cracks predicted fatigue cracks over 20 years.
- Crack prone, greater than 40 percent predicted fatigue cracks over 20 years.

These categories of cracking were used to segregate the different AC base mixtures because estimating from the job mix formula data does not represent the actual mixture that was placed along the roadway of each project – only an estimate of the volumetric properties. Table 6-1 summarizes the percentage of the Ohio 301 and 302 specifications found to be crack resistant or crack prone. It should be noted that these values only represent the projects that were selected for this study. Shown in Table 6–2 through Table 6–4 are a listing of all projects and their associated ranking based on the predicted bottom-up cracking values at 20 years for ODOT Item 301, ODOT Item 302, and unknown ODOT Item, respectively. "NA" was used to identify those AC base mixtures with insufficient mixture design information to complete the adjustment procedure for the bottom-up fatigue cracking coefficients.

Table 6-1. Summary of Performance Ranking based on Cracking Potential.

	Item 301	Item 302	Unknown
Percent of selected projects that are crack resistant	40%	33%	43%
Percent of selected projects that are crack prone	10%	29%	0%
Total number of selected projects	10	24	7

Table 6–2. Summary of Performance Ranking Predicted using ME Design Software for Bottom-Up Fatigue Cracking, ODOT Item 301.

Project ID Number	Performance		
	Crack Resistant	Average	Crack Prone
FRA/LIC-161-23.20/0.00	A		
LIC-161-1.83		B	
CLI-73-6.52			C
GAL-35-8.32		B	

Project ID Number	Performance		
	Crack Resistant	Average	Crack Prone
ROS-207-0.00	NA	NA	NA
WAY-30-11.86 "301"		B	
FAI-33-17.44 SR 13	A		
MER-219-14.04 "301"		B	
FAI-33-13.25	A		
FAI-33-17.44	A		
MAD-142-0.49	NA	NA	NA
VIN-50-11.75		B	
Totals for ODOT Item 301	4	5	1

Table 6–3. Summary of Performance Ranking Predicted using ME Design Software for Bottom-Up Fatigue Cracking, ODOT Item 302.

	Performance		
Project ID Number	Crack Resistant	Average	Crack Prone
FRA-71-5.29	A		
FRA-270-21.67		B	
FRA-270-35.41 Part 2	NA	NA	NA
HAN-30-3.00	NA	NA	NA
LUC-75-6.70		B	
WOO-75-2.37		B	
WOO-75-19.43			C
ATH-33-11.74	A		
CUY-71-0.00	A		
CLA-70-13.98		B	
DEL-23-17.64	A		
LAK-2-7.60		B	
CLI-73-12.03	NA	NA	NA
BUT-75-5.91	A		
DEF-24-7.96			C
FRA-70-3.41		B	
SUM-77-17.20		B	
HAN/WOO-75-19.22/0.00			C
WAY-30-11.86 "302"	A		
FAI-33-19.79	NA	NA	NA
MER-219-14.04 "302"			C
ASD-39-0.00			C
FAI-33-7.31	NA	NA	NA
FAI-33-19.79 SR 18	NA	NA	NA
LUC-2-21.24			C
WAR-71-14.20			C
FUL-20-10.86	NA	NA	NA
WOO-795-2.01		B	
CUY/SUM-77-0.00/32.73	A		
COS-36-20.83	A		
DEL-23-19.24		B	
Totals for ODOT Item 302	8	9	7

Table 6–4. Summary of Performance Ranking Predicted using ME Design Software for Bottom-Up Fatigue Cracking, unknown ODOT Item number.

	Performance		
Project ID Number	Crack Resistant	Average	Crack Prone
MED-42-17.68		B	
WOO-75-10.61	A		
MRW-71-3.17	A		
WAR-71-3.78		B	
MED-71-9.56		B	
FAY-35-2.57	NA	NA	NA
GRE/FAY-35-26.20/0.00		B	
HAM-264-6.87	A		
Totals for unknown ODOT Item	3	4	0

An overall comparison was made between the performance prediction categories and the categories of different mixture properties measured in the laboratory of the cores recovered from each project, two of which are noted below.

- Table 6–5 includes a comparison of the fatigue cracking resistant categories and the average flexibility index from the SCB test category. As shown, no correlation was found between the two categories. (NOTE: the cells in Table 6-4 that are not highlighted represent the same mixture observation.) A couple of reasons for this observation is that the flexibility index is age dependent and the SCB test results are probably heavily dependent on the size of the aggregate and/or gradation of the mixture.
- Table 6–6 includes a comparison of the fatigue cracking resistant categories and the average dry indirect tensile strength category. As shown, there is a reasonable correlation between the fatigue cracking category and indirect tensile strength category. A couple of reasons for this observation is the dry indirect tensile strength was found to be independent of mixture age and the indirect tensile strength is less dependent on aggregate size. More importantly, gradation is taken into consideration when determining the fatigue cracking fracture coefficients.

Table 6–5. Comparison of the Flexibility Index Category and Fatigue Cracking Category; Number of AC Base Mixtures.

Flexibility Index	Crack Resistant	Average	Crack Prone
>1.5	5	8	5
0.75 to 1.5	4	6	3
<0.75	6	3	0

Table 6–6. Comparison of the Dry Indirect Tensile Strength Category and Fatigue Cracking Category; Number of AC Base Mixtures.

Dry Tensile Strength Category	Crack Resistant	Average	Crack Prone
>1400	12	4	0
1000 to 1400	2	11	1
<1000	1	3	7

7 Research Findings and Conclusions

A total of 728 cores were extracted from 51 project sites across the state of Ohio. These cores were tested to determine cracking potential, moisture susceptibility, durability, and density. Information gathered from the corresponding JMF(s), where available, were used to estimate coefficients necessary for the estimation of rutting and fatigue cracking. Predicted truck traffic, which would have been used to design pavement thickness, was compared to the most recent measured truck traffic count. The performance of each section, in terms of PCR, was obtained, when available, from ODOT. The findings and conclusions from these analyses are summarized below.

Based on the analysis of pavement condition and the district survey:

- Nine projects were identified as having poor performance based on the following criteria:
    - Five of the projects performed poorly based on PCR history. One of these was also identified by the District as performing poorly. All but one was overlaid within 15 years of service.
    - The Districts identified two additional projects requiring an overlay prematurely.
    - An additional two projects were included because they were overlaid within 15 years of service.
    - 7 of the 9 sections exhibited wheel track cracking.

Based on samples collected from the project sites:

- At eight project sites, segregation was observed by the research team during base construction or in the collected cores.
- Although core height closely matched planned thickness, on average, approximately 20% of cores were at least 1 in (25 mm) deficient.

Of the nine projects identified with poor performance:

- Three projects had a high actual to predicted truck traffic ratio, which likely explains their performance, as they were subjected to higher truck traffic volumes than they were designed for.
- One was an experimental section incorporating a stone matrix asphalt (SMA) overlay. The poor performance of this section can be attributed to the poor performance of the SMA.
- Five of the nine projects were specified in the plans as ODOT Item 880. This accounts for 38% of projects specified as Item 880.
- The AC base for two of the nine projects were constructed with Item 301. This represents approximately 17% of projects constructed with Item 301 included in this study.
- Five projects were constructed with Item 302 which represents approximately 16% of projects constructed with Item 302 included in this study.
- One project was constructed with both Item 301 and Item 302 and for one of the nine projects the mix type for the AC base was not known.
- The percentage of projects included in this study constructed with Item 301 which had poor performance were approximately equal to the percentage of projects constructed with Item 302 which had poor performance.

- The number of projects with poor performance is too small to draw conclusions regarding the performance of pavements constructed with Item 301 or 302 AC base mixes. Project performance could be due to numerous reasons and may not be due to asphalt base layers.

Of the seven projects identified with segregation in the samples:

- Three had high Cantabro mass loss (> 30%), low TSR (0.51 or less), and high average in-place air content (> 7%).
  - All three projects were constructed under ODOT supplemental specification (SS) 880.
- One project was constructed with Item 301 AC base, two were constructed with Item 302 AC base, one project was constructed with both Items 301 and 302 AC base, and the mix type used for AC base was unknown for the remaining three projects.

Five projects were defined to have exceptional performance based on PCR history.

- The laboratory test results for cores extracted from project sites with exceptional performance were mixed:
  - Three had average FI less than 1.0.
  - Three of the five had high Cantabro mass loss.
  - Two had a TSR less than 0.70, both were 0.64.
  - Four of the five project sites with exceptional performance had AC base with average in-place air voids between 4.0% and 6.5%, while the fifth had average in-place air voids of 8.0%.
  - Cores taken from project sites in the exceptional performance category had the lowest in-place air voids, averaging 5.8%
- Three of the five projects were constructed with Item 302 AC base; one was constructed with Item 301 and for one project, the mix type used for the AC base was unknown.

Based on the laboratory testing performed for Phase 1:

- The following are trends observed relative in-place air voids of the AC base mixes:
  - In-place air voids of the top AC base lift ranged from 1.1% to 14.4%
  - Average in-place air voids were generally higher in SS 880 than in ODOT Item 301 and Item 302. The trend however was not significant.
  - Several project sites had a very large spread for in-place air void which is a sign of variability, and possibly segregation.
  - AC bases from project sites with poor or average field performance tended to have greater spread of in-place air voids.
- Large aggregate in the mix influenced the fracture path (crack band) during the semicircular bend (SCB-IL) test, resulting in large variability in the data. However, the data did exhibit some trends such as:
  - As expected, flexibility index (FI) generally decreased with increasing specimen air voids. The trend was not significant with an R^2 of 0.06. The same insignificant trend was observed for fracture energy and specimen air void content with an R^2 of 0.29
  - As expected, sections with poor performance were more likely to have a low flexibility index (FI).
  - There is a lot of variability in the results from the SCB test and the test results were found to depend on mixture age. In general, older AC base mixes tended to have lower FI. Thus age is a confounding factor which needs to be considered through

short and/or long-term aging if the test is to be used for future work and in specifying AC base mixtures.
  - In comparing average FI among pavements 10 years or older, the mean FI among projects constructed with Item 302 AC base was 1.8 times greater than the mean FI among projects constructed with Item 301 AC base.
  - Limestone AC base mixes 10 years or older tended to have higher FI values than gravel or blended aggregate AC base mixes in the same age range. Eight of the fourteen limestone mixes had FI greater than 1.0. For gravel and blended aggregate AC base mixes in the same age group, the majority had FI values less than 1.0.
  - It is recommended the SCB-IL test not be used to evaluate asphalt base at this time due to the high variability observed in the test results. Further investigation of the effect of large aggregate on SCB-IL test results is needed before this test can be used for acceptance of AC base. Based on test specimen examination, addressing the segregation of base mixes, and providing more control on in-place air voids will likely reduce the variability of the test results.
- The following are trends observed in the Tensile Strength Ratio (TSR) data:
  - As expected, the tensile strength of the conditioned specimen and TSR decreased with increasing average in-place air voids, although the relationships were weak.
  - The majority of project sites with poor field performance had TSR values for AC base mixes less than the 0.70 criterion.
  - Only 25% of the SS 880 specimens met the TSR criterion of 0.70.
  - Of the project sites which used Item 302 base mixes and were tested for TSR just under 50% met or exceeded the 0.70 criterion, whereas 30% of the project sites with Item 301 base mix passed the criterion
- The following are trends observed in the Cantabro test data:
  - A moderate relationship exists, $R^2 = 0.42$, between specimen air voids and Cantabro mass loss, with mass loss increasing with increasing specimen air void content. Additional testing may identify the cause for outliers which would improve the relationship.
  - Of the 13 new pavement sections, 85% had AC Cantabro mass loss less than 16% for the tested AC base mix.
  - There is a weak relationship, $R^2 = 0.20$, between FI and Cantabro mass loss, with FI decreasing with increasing mass loss. It was also observed for specimens with a mass loss of 40% or more, FI values were less than 2.1. It is recommended the relationship between FI and Cantabro mass loss be further investigated.
  - A moderate relationship was found between Cantabro mass loss and TSR, indicating an increase in mass loss is related to a decrease in TSR. Furthermore, it was found AC base mixes with mass loss greater than 21% tended to have TSR values less than the established criterion of 0.70.
- It is well known asphalt binder content plays an integral role in fatigue resistance and mix durability. Based on the review of all JMFs, Item 302 AC base mixes tend to have lower binder content than Item 301 AC base mixes. However, due to multiple JMFs approved on a project, the effect of binder content on laboratory testing, field performance, and in-place air voids could not be evaluated in this study. It is recommended the effect of binder content be investigated in Phase 2.

Based on the comparison of specifications from ODOT and other state DOTs:

- The minimum temperature of the mix at the paver specified by ODOT, 250°F (121°C), is on the low end of temperatures specified by other states reviewed.
- Four of the seven states with an asphalt base specification include a density acceptance criterion.
- Pennsylvania DOT, which has an asphalt base specification, requires total pavement thickness be within 0.5 in (13 mm) of the specified value.
- Michigan DOT and Texas DOT include specifications to identify and control segregation.

Based on the ME analysis:

- Air voids, asphalt content, and gradation are important volumetric properties which have an impact on fatigue strength of AC mixtures. Air voids and asphalt content are the more important and design asphalt content is related to the gradation of the mixture. The design air voids and target asphalt content have a significant effect on the fatigue cracking coefficients in accordance with the ME Design software. As such, it is recommended air voids (percent compaction) or mixture density be used or considered for future work to improve Ohio's AC base mixture specification.
- The dry indirect tensile strength was found to be related to the fracture or fatigue strength coefficients. As such, the dry indirect tensile strength should be considered for use in designing and/or specifying AC base mixtures.

8 Recommendations for Implementation of Research Findings

Based on the findings reported above, the research team identified the following opportunities to improve the quality of ODOT asphalt base:

- Segregation was seen in asphalt mat and cores. Phase 2 should include an investigation of methods to identify segregation during paving.
- Measured in-place air voids were highly variable. Phase 2 should include an investigation to determine if there is a need for a density specification.
- Current practices do not adequately control pavement thickness. Phase 2 should include an investigation of other methods to ensure as-built thickness is adequate.
- A threshold value for Cantabro mass loss for ODOT AC base material should be investigated further.
- Based on the review of specifications from other states, it is recommended delivery and compaction temperature of the asphalt base be monitored in Phase 2 to evaluate the impact on in-place air voids and segregation.

9 Phase 2 Research Plan

The objective of Phase 2 is to sample and test material from four active asphalt base construction projects in accordance with proposed specifications to determine whether proposed changes would impact the mix design or construction process and improve the quality of asphalt base construction. Four to six projects would be chosen to test both ODOT CMS Item 301 and Item 302 material, test material with low and high binder content, and have a range of RAP (and RAS if available) content. If the timing is appropriate, the SOLVER test road in Vinton County can be included as one of the sections to reduce sample collection and testing costs.

The objectives for Phase 2 will be met via the following tasks:

Task 1. Monitor construction process:
The JMF used at the project site will be obtained from which pertinent mix design information, including asphalt binder content, RAP/RAS content, aggregate type, and compaction temperature will be collected. Information related to the paving and roller equipment such as make and model will be documented. Construction will be monitored and pertinent information will be collected, including temperature of the AC base mix in the paver hopper and at compaction, roller pass pattern, air temperature, paving, distance between paver and roller, etc.

Task 2. Collect material samples:
Sufficient amounts of AC base mix will be sampled from the plant to determine maximum specific gravity of the mix and asphalt binder content by ignition oven and to complete laboratory testing, including Cantabro mass loss, indirect tensile strength, and tensile strength ratio. Laboratory testing will performed on both laboratory compacted and field compacted specimens, therefore additional cores will also be extracted.

Task 3. Measure asphalt base density:
The density of each lift of asphalt base will be determined using the procedure described in ODOT CMS Item 446 except cores will not be extracted from the longitudinal joint. Cores will be collected from four production days, or lots, of paving. The measured in-place air content will be determined. Additionally, ground penetrating radar (GPR) will be used for measuring in-place density during construction. Results will be compared with density measured from extracted cores.

Task 4. Measure asphalt base segregation:
The extent of segregation will be determined and evaluated for the selected sections using the following three procedures. In addition to measuring temperature in the paver hopper and at compaction, the following tests will be completed:

1. TXDOT method Tex-207-F, Part V
2. TXDOT method Tex-244-F
3. Michigan DOT 2012 Standard Specification for Construction Section 501.

Task 5. Measure asphalt base and pavement thicknesses:
Cores will be extracted from the pavement after placement of the surface. The thickness of these cores, as well as the cores extracted from the asphalt base in Task 1 will be measured and evaluated using Pennsylvania DOT Specification Section 309 and Pennsylvania DOT Test Method No. 737. Other methods such as ground penetrating radar (GPR) will be considered. The variation of asphalt base and total pavement thickness will be evaluated.

Task 6. Measure asphalt base TSR:
Indirect tensile strength tests will be performed to determine TSR for laboratory compacted specimens and extracted cores. The results will be compared to ODOT's TSR criterion of 0.70. Comparisons will be drawn between TSR determined for both the lab and field compacted specimens. Additionally, TSR determined from the mix design for Item 302 material will be compared with TSR determined for both the lab and field compacted specimens.

Task 7. Analyze data:
The researchers will compare and contrast the collected data with the acceptance criteria in the above specifications for density, segregation, and thickness. The variability of these parameters will also be evaluated with respect to CMS Item, RAP/RAS, binder content, etc. Laboratory test results will be analyzed to evaluate the effect of asphalt binder content on durability (as measured by Cantabro mass loss), and TSR. Additionally, resistance to area fatigue cracking will be predicted using procedures based on mix volumetric parameters (see Appendix M).

Task 8. Prepare final report:
Four months before the completion date of the project, the researchers will submit a draft final report documenting the work performed in Task 1 through Task 7 and providing recommendations, including any changes to ODOT specifications.

The researchers request ODOT or the project contractor provide the following assistance:

- Provide access to sites for collecting material specimens and data.
- Fill coring holes after specimens are collected.
- Provide construction records and plans for each project.
- Perform nuclear density testing in accordance with TXDOT method Tex-207-F, Part V, and Michigan DOT 2012 Standard Specification for Construction Section 501.
- Perform thermal imaging in accordance with TXDOT method Tex-244-F using a paver mounted system.

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11 Appendices

Appendix A: Literature Review

Durability

Durability of asphalt mixtures and pavements has long been a topic of concern. Lee [1969] indicated durability is often defined as “its resistance to weathering, aging, and traffic loading, or as the ability to resist change due to these destructive or deteriorative forces.” More recently, Nicholls et al. [2008] stated durability of an asphalt material or pavement is generally defined by its ability to retain structural integrity and to serve functionally at a satisfactory level under environmental effects and expected traffic loading throughout its design life. Specifically, Nicholls et al. [2008] offer the following definitions for asphalt durability and pavement durability:

- Asphalt durability: “Maintenance of the structural integrity of compacted material over its expected service-life when exposed to the effects of the environment (water, oxygen, sunlight) and traffic loading”
- Pavement durability: “Retention of a satisfactory level of performance over the structure’s expected service-life without major maintenance for all properties that are required for the particular road situation in addition to asphalt durability”

According to Bonaquist [2014], in order for a flexible pavement to be durable it must be structurally adequate, properly drained, properly constructed, and built with durable materials. The pavement must have sufficient thickness to carry the expected traffic loading and protect the underlying subgrade. Proper drainage is necessary to minimize infiltration of water into the pavement structure. Good construction practices should be followed, including proper grading and compaction of unbound layers, ensuring a bond is achieved between bound layers, proper control of layer thicknesses, compaction, and minimal segregation. The pavement must be built with durable materials which can withstand the effects of aging, traffic, and the environment. Specifically, unbound layers should be resistant to moisture and frost, while the asphalt concrete must be resistant to aging and moisture effects as well as be able to withstand traffic and environmental loads. [Bonaquist, 2014]

In the context of this study, the focus has been placed on asphalt durability. Bonaquist [2014] conducted a comprehensive review of factors influencing durability of AC mixes. While the focus of that paper was primarily for durability of AC surface mixes and the impact of mix design on durability, the factors influencing AC mix durability are applicable to all courses of asphalt mix, and thus pertinent information from that paper is summarized first.

Bonqauist [2014] identifies the factors which affect asphalt mix durability: environment, drainage, construction, and mixture composition. Bonaquist [2014] notes construction of the asphalt concrete can have a significant effect on durability of a mix and may result in localized defects and distresses, while deficiencies related to mixture composition are generally widespread.

Two primary environmental effects were noted: temperature and moisture. Asphalt concrete is known to be sensitive to temperature, which affects stiffness, rutting resistance, and cracking resistance, therefore temperature is a primary consideration in design. Moisture can damage asphalt concrete mixes in three ways: “loss of cohesion within the asphalt binder or mastic; loss of adhesion between the asphalt binder and the aggregate; and aggregate degradation particularly when freezing occurs in the mixture." Drainage in both the surface and subsurface are essential to prevent water from penetrating the interconnected voids within the asphalt concrete and becoming trapped, leading to moisture damage. [Bonaquist, 2014]

In regards to construction, weather conditions, segregation, compaction, joints, and layer bonds are all factors in construction which impact AC mix durability [Bonaquist, 2014]. Weather conditions such as temperature, moisture, and wind can impact the time available for compaction, and the degree of compaction, as measured by in-place air voids. Temperature and moisture influence the bond between layers, and an adequate bond is necessary to ensure structural integrity of the pavement. Bonaquist [2014] further points out that although warm-mix asphalt (WMA) allows for compaction at lower temperatures, the underlying layer must have sufficient heat to achieve an adequate bond between layers. As Bonaquist [2014] notes, segregation significantly affects AC mix durability and is a common construction problem. Segregation has been defined as localized areas of coarse materials in some places in the mat and fine materials in other areas in the mat [Stroup-Gardiner and Brown, 2000]. This creates a non-uniform mat, with mix properties varying in areas with coarse and fine materials. Coarsely segregated areas typically have lower asphalt content, and higher air voids, and have been found to have lower tensile strength and shorter fatigue life [Stroup-Gardiner and Brown, 2000]. Whereas, finely segregated areas typically have higher asphalt content and lower air voids [Stroup-Gardiner and Brown, 2000]. Compaction is arguably the most important factor related to performance [Bonaquist, 2014]. Due to the importance of in-place air voids, the topic of compaction will be discussed in more detail in a later section. Bonaquist [2014] also discusses the importance of joints as both longitudinal and transverse joints create areas of weakness in the pavement due to often lower air voids, increased permeability and segregation at the joint. As noted previously, adequate bond between layers is necessary, as failure to do so may lead to slippage of layers.

Lastly, Bonaquist [2014] notes the properties of the individual components of the mix as well as the selected gradation and resulting volumetric properties of the mix also impact AC mix durability. While the mixture composition has a significant effect on durability [Bonaquist, 2014], a substantial body of work exists on the topics of aggregate and binder properties, gradation and volumetric properties. Due to the breadth of the topic of mixture composition, the focus for this literature review will be placed on testing of the mix within the context of an asphalt base layer.

Various laboratory tests have been used to evaluate durability of asphalt concrete mixes. Typically, for AC mixes, tests are aimed at assessing the susceptibility to moisture, rutting, cracking, and, more recently, disintegration. For AC base mixes, rutting potential is not of concern due to its location below the surface of the pavement structure. A test commonly used for assessing moisture susceptibility is indirect tensile strength and tensile strength ratio (TSR). Several tests exist to assess crack resistance, such as the disk-shaped compact tension (DCT), various test methods for the semi-circular bend (SCB) test, bending beam fatigue test, Texas Overlay tester, and others. Discussion on moisture damage and crack resistance is provided elsewhere in this report. Cantabro testing has been used with increasing frequency in the last decade to evaluate durability of dense graded AC mixes.

Cantabro Test

The Cantabro test is a fairly simple test and can be completed in approximately 40 minutes for laboratory compacted samples [West, 2017]. The test consists of placing an AC pill into the Los Angeles Abrasion drum without the steel charges and subjecting the pill to 300 revolutions. The weight of the pill is measured before and after the test from which the mass loss (ML) is determined. Typically the test is run at 77°F (25°C) and the dimensions of the pill and desired sample air voids are identified for the specification. Currently national specifications include AASHTO TP 108 and ASTM D7064, while several states have their own standard for Cantabro testing [Cox et al., 2017]. During the test, material becomes dislodged from the pill, indicating the material's susceptibility to disintegration. Therefore, low values of ML are desired. Values in the range of 15-30% have been used as criteria for open graded surface mixes, with the higher end of the range associated with aged mix [Cox et al., 2017]. Cantabro testing has traditionally been used to determine the minimum asphalt binder content needed for open graded or porous surface mixes. However, due to its simplicity, it has gained attention as a method for evaluating durability of dense graded mixes. Currently no criteria exists for ML of dense graded AC mixes.

A group of researchers at Mississippi State University (MSU) began conducting research in 2009 to evaluate the feasibility of using the Cantabro test to evaluate dense graded AC mixes [Cox et al., 2017]. Although research began at MSU in 2009, Cantabro testing had been conducted on dense graded asphalt mixes prior to that. For example, Cantabro testing in addition to Marshall stability and indirect tensile stiffness modulus testing was conducted on dense graded mix as part of a study published in 2008 on the use of waste plastic in bitumen [Aschuri et al., 2008]. Aschuri et al. [2008] measured the ML after discrete intervals of revolutions, subjecting the samples to a total of 500 revolutions.

The research group at MSU has published several studies on the topic of Cantabro testing for dense graded mixes in which the effects of field aging, laboratory aging, RAP content, and production as WMA were investigated. Cox et al. [2017] have summarized key findings from these studies, as presented in Table 11-1. It should be noted mixes varied in terms of pavement layer and the focus of these studies was not necessarily on AC base mixes, however many of the key findings are applicable to all AC mixes regardless of where in the pavement they are placed.

Table 11–1. Summary of Cantabro Testing of Dense Graded AC Mixes by MSU and Collaborators [adapted from Cox et al., 2017].

Reference	Key data and goals	Key findings
Doyle and Howard, 2010	RAP 0 -100%, includes data used in other publications	• Air voids, RAP content, and conditioning observed to affect Cantabro ML • ML for 25% RAP mixes were similar to control mixes; 50% RAP had ML values marginally higher • Includes ML value from Mississippi DOT QA specimens
Doyle and Howard, 2011	Seven mixes with modest data set. Served as pilot effort to assess Cantabro test	• ML sensitive to asphalt content and air voids; all ML values less than 15% • Use of Cantabro for dense graded mixes found to be promising
Doyle et al., 2011 and Mejias-Santiago et al., 2012	Relative durability of HMA and WMA for mixes containing up to 50% RAP using PG 67-22 binder	• Appreciable increase in ML for asphalt contents less than design asphalt content, but not as much for contents greater than design • No statistical effect of WMA on ML • Effect of aggregate type and RAP found to affect ML
Baumgardner et al., 2012	Mixing efficiency for ground tire rubber (GTR) and additive were	• Assessed open graded and dense grade AC mixes with Cantabro testing; concluded SBS-modified PG 76-22 and GTR wet-processed binders outperformed GTR

Reference	Key data and goals	Key findings
	assessed for open graded and dense graded AC mixes	dry-processed binders
Howard et al., 2013	Dense graded mixes with very high RAP content (50% to 100%) compared with mixes with conventional RAP content (15%)	• Based on Cantabro results concluded 20% to 35% RAP was more feasible for many projects than high RAP on a few projects • Found ML and indirect tensile strength are related for unaged mixes tested at 25°C (77°F)
James, 2014	Doctoral dissertation on airfields; Marshall mix design compared with Superpave design	• ML increased with air voids due to water conditioning and when unmodified used in place of polymer modification • Finer mixes had lower ML than coarser mixes
Howard and Doyle, 2015	First study looking at field-aging effects compared with laboratory conditioning	• One-year field aging in Mississippi showed fairly linear relationship between air voids and ML • Laboratory long term aging (R30) produced less damage than one year field aging • 28 days of aging at 60°C (140°F) approximately simulated one year of field aging
Doyle and Howard, 2016	Plant mix samples and long term condition evaluated with goal to assess fundamental properties of laboratory use of Cantabro test	• ML increased 2 to 4% due to long term conditioning (R30) which was 0 to 1% more than ML for 168 hours at 64°C (147°F) • Errors from testing 3 specimens (versus 30) were manageable at 0.9% to 1.5% (i.e. ML differences greater than 1.5% were meaningful and ML differences less than 0.9% were associated with testing) • Sensitivity and multiple regression assessment resulted in equation that relates ML to performance grade, gravel aggregate percentage, air voids and effective binder (VBE) • Variability assessment found no significant ML difference when 0.2% binder was added, but mean ML was 0.4% lower for otherwise identical mix with 0.2% more binder

The study conducted by the MSU group on RAP-dominated WMA investigated base layer AC mixes (i.e. non-surface mixes) [Howard et al., 2013]. In that study, high RAP (50% to 100%) WMA was compared with control hot-mix asphalt (HMA) containing 10% to 15% RAP. The nominal maximum aggregate sizes (NMAS) used for the RAP-WMA mixes were 12.5 mm (0.5 in) for 50% and 75% RAP and 9.5 mm (0.375 in) for 100% RAP. Similarly, the control HMA mixes consisted of NMAS of 9.5 mm (0.375 in), 12.5 mm (0.5 in), and 19.0 mm (0.75 in). As part of the study, Cantabro testing was conducted on unaged samples of control and RAP-WMA mixes. Control HMA mixes consisted of laboratory mixed and laboratory compacted mix, as well as plant mixed and laboratory compacted mix, and plant mixed and field compacted mix. Air voids for the Cantabro samples ranged from 4.1% to 5.2%. ML for RAP-WMA mixes generally increased with RAP content, such that ML for 50% RAP-WMA mixes ranged from 13.4% to 15.8%, ML for 75% RAP mixes ranged from 18.1% to 21.4% and ML for 100% RAP mixes ranged from 17.0% to 31.8%. Results for the control mixes were lower, with mean ML for the 12.5 mm (0.5 in) and 19.0 mm (0.75 in) NMAS mixes of 9.8% and 10.6%, respectively. The authors concluded RAP dominated mixes were more prone to ML in Cantabro testing than typical Mississippi DOT mixes. [Howard et al., 2013]

West et al. [2017] evaluated four laboratory performance tests against measured cracking performance in the Federal Highway Administration's (FHWA's) accelerated loading facility (ALF). The four laboratory tests included in the study were Cantabro test, SCB test, indirect tensile test (IDT) and a modified version of the overlay test. Ten mixes were included in the study with all of them sharing the same 12.5 mm (0.5 in) NMAS gradation design and gyration level. The mixes varied by the type, or lack thereof, of WMA technology, and by the use and content level of RAP or RAS. Two lifts of each mix were placed atop a thick aggregate at FHWA's ALF. In comparing ML with cracking measured in the ALF, the authors indicated a moderate relationship (R^2 = 0.54 to 0.59) existed between ML and the number of passes to reach 20 ft. (6.1 m) of cracking in the ALF. The authors reported higher ML was associated with fewer passes to reach 20 ft (6.1 m) of cracking and that ML greater than 7% appeared to separate the worst fatigue cracking performance.

Performance of asphalt pavement

Common types of asphalt pavement distress include cracking due to fatigue, temperature cycles, or aging; disintegration due to moisture and/or freeze-thaw cycles; and rutting due to repeated loading and/or weak foundation.

Low Temperature Cracking

For years, researchers have evaluated the low-temperature cracking of asphalt mixtures as one of the most common distresses in asphalt pavements. In each study different temperatures, loading rates and modes, have been used during the performance of tests such as Disk-Shaped Compact Tension (DCT) and Semi-Circular Bending Test (SCB). In many of these studies, the fracture energy has been the main focus of analysis.

Marasteanu et al. [2007] found the field performance of the mixtures correlated best with the estimations of fracture toughness and energy calculated using the SCB and DCT tests, than with parameters obtained from other more traditional tests such as Indirect Tensile Test (IDT) applied to mixtures or Bending Beam Rheometer BBR and Direct Tension Tester DTT applied on asphalt binder. Hence, the selection of fracture resistant binders and asphalt mixture should be governed by their response to fracture mechanic-based tests [Marasteanu et al., 2007].

Saadeh and Hakimelahi [2012] also concluded the SCB test has a considerable potential as a QA/QC test of fracture properties of asphalt mixtures. They successfully modeled the SCB test and the crack propagation in Finite Element [FE] software, where they found the mechanism of failure of the SCB test was mainly attributed to tensile stress [Saadeh & Hakimalahi, 2012]. At the same time Walubita et al. [2013] found that at room temperature the SCB test exhibited high variability (COV > 30%) and the DCT test became problematic, regarding the loading mode (monotonic or dynamic loading). Besides the SCB high variability, the researchers also observed the process of specimen fabrication for the DCT test was very tedious. Overall, it was suggested these tests are impractical for daily routine HMA mix-design at room temperature and appear to be a better suited for research-level at low temperature, and for low AC mixes [Walubita et al., 2013].

IL-SCB Test

After conducting different tests such as SCB, DCT, IDT, dynamic modulus test (E*), Texas overlay test (TOL), and push-pull fatigue test, to evaluate the properties and performance of AC mixtures with up to 60% RAP and RAS, Al-Qadi et al. [2015] found none were able to accurately and consistently predict and rank cracking resistance of an AC mix. However, the potential low-cost implementation, equipment availability, and the relative ease of specimen preparation and testing of the low-temperature SCB test made it the protocol considered for further analysis. Al-Qadi et al. [2015] then developed a modified SCB procedure: the IL-SCB test (or AASHTO TP-124) which used intermediate testing temperatures and introduced a Flexibility Index (FI) which would make the IL-SCB more consistent.

The fracture potential and FI in the IL-SCB method were validated using plant-produced, laboratory-produced, and field core specimens to predict the cracking resistance among mixes. It was also supported by finite element simulations. It was found intermediate temperatures allowed a clear distinction between mixes, which was easily masked by experimental variability at low temperatures. On the other hand, the introduced FI provides a means to identify brittle mixes which are prone to premature cracking. For instance, an increase in the RAP or RAS content showed a reduction in the FI, which indicates a more brittle behavior. For wearing surface mixes with nominal maximum aggregate size 0.75 in (19 mm) or less, the researchers found with some exceptions, good performing sections had an FI greater than 10 and poor performing sections had an FI less than 6. [Al-Qadi et al., 2015].

This relationship between the percentage of RAP and the behavior of the mixture agreed with results previously obtained by West et al. [2013] and Wagoner [2005]. West et al. [2013] found mixtures with high RAP content developed lower fracture energy than the control samples (without RAP). On the other hand, Wagoner et al. [2005] found after conducting a DCT test, that in general, lower fracture energy was found to be related to a decrease in the temperature, which at the same time appeared to change the behavior of the mixture from quasi-brittle fracture with softening response to brittle fracture with minimal softening after peak [Wagoner et al., 2005].

This relationship between fracture energy and temperature was also observed by Marasteanu et al. [2007] who concluded a change in temperature could change the behavior of the mix from brittle-ductile to brittle. Later on, Williams et al. [2011] performed a DCT test on mixtures with and without recycled material, finding as the recycled material content increased the fracture resistance decreased. It was also found that to achieve a minimum fracture energy of 375 J/m2 (0.033 BTU/ft2) the percentage of binder replacement should not exceed 35%. These findings are similar to those obtained by Al-Qadi et al. [2015] using the the IL-SCB test together with the Flexibility Index (FI).

Other findings on Low-Temperature Cracking

Results from Marasteanu et al. [2007] also showed a loading rate dependency of the asphalt mixture. However, it was recommended further evaluation be conducted to improve its match with field cooling rates. Previously, Wagoner et al. [2005] had found that in fact, as the loading rate increased the fracture energy decreased. Walubita et al. [2013] also studied the effect of the loading, in that case the two loading modes: monotonic and dynamic loading. They found that while monotonic loading tests were simpler, less time consuming and more repeatable, they were not as good as the dynamic loading tests on capturing variations on design parameters (asphalt content and temperature) and the difference between mixes.

Wagoner et al. [2005] results also suggested the variations on temperature affected the crack path and the thickness of the specimen used for the DCT test had an effect on the fracture energy. Later on, Saadeh and Hakimelahi [2012] compared the SCB test against the Beam-fatigue test (BFT). The authors found a better correlation among most of the parameters from both tests than between them and the initial stiffness, which suggested the initial stiffness is not a good indicator of fracture properties.

Van Deusen et al. [2015] used the DCT test to evaluate the thermal fracture properties of asphalt mixtures with different construction practices and mix design characteristics. They found a significant drop in fracture energy from the mix design to production, but no specific cause was identified. Researchers also noticed mixtures with polymer modified binder reached the required fracture energy (400 J/m2 (0.035 BTU/ft2)) while those with unmodified binder did not [Van Deusen et al, 2015]. Other authors such as Marasteanu et al. [2007] have already found physical hardening was not only important for binder properties, but also key for fracture and bending beam rheometer (BBR) test performance. In fact the researchers recommended updating the AASHTO M320 standard to account for the improved fracture properties of polymer modified binder. In the same study, the mixture coefficient of thermal contraction was found to be a critical parameter for predicting low temperature cracking [Marasteanu et al., 2007].

Marasteanu et al. [2012] investigated the thermal stresses and physical hardening effects of warm mix asphalt (WMA) and mixtures modified with polyphosphoric acid (PPA) and RAP. After evaluations, an improved model which considered these effects was proposed. It was found that using different types of polymer will not significantly change the rate of physical hardening, which depends on the conditioning temperature and source of base binder. Moreover, the rate of physical hardening affects the creep response at temperatures below and near the glass transition [Marasteanu et al., 2012]. The authors also pursued the idea of an improved thermal cracking (TC) model which considers the fracture mechanics properties obtained with SCB and DCT tests and the thermal fatigue at low temperatures. As a result, a new thermal cracking model, the ILLI-TC, was developed and validated. It was found to be more accurate than the TCMODEL on quantifying the cracking mechanisms in pavement [Marasteanu et al., 2012]

Dave et al. [2015], also using the DCT test, found the increase of the fracture energy of the mixture and the process of reclamation of existing pavement prior to overlay significantly improved the cracking performance of the pavement. It was also found volumetric control measures may not be enough to control the thermal cracking performance of a mixture. Researchers also concluded that while the low-temperature binder grade has a significant effect on the cracking performance, the total binder content and recycled binder content were not comparable with the cracking amount [Dave et al., 2015]. Urquhart [2016] found the Dynamic Shear Rheometer (DSR) stress ratio showed a significant correlation with the fatigue life, and suggested DSR test is appropriate to rank the low-temperature cracking performance of all binder grades in Australia.

Moisture Damage

Moisture can cause stripping in the mix which can be detrimental to the AC pavement. The moisture susceptibility is vital in characterizing the performance of the mix. Moisture susceptibility is measured by subjecting an asphalt specimen to accelerated saturation combined with a freeze-thaw cycle. The conditioned (wet) Indirect Tensile Strength and the non-conditioned (dry) Indirect Tensile Strength are combined to obtain the Tensile Strength Ratio (TSR). Many agencies and researchers have suggested or adopted a minimum value of the TSR as a value that assures a certain resistance to moisture damage in the AC mixture. For instance, the Ohio Department of Transportation (ODOT) requires a TSR greater than 80% for ODOT Item 442 mixes and greater than 70% for all other mix types.

Some authors such as Khosla and Harikrishnan [2005] have suggested other factors should be considered in addition to the TSR value in determining moisture susceptibility. They found mixtures with an acceptable TSR value (85%) and a high indirect tensile strength may perform better than mixtures with higher TSR but lower indirect tensile strength. Thus considering TSR alone to evaluate the moisture susceptibility may be misleading [Khosla and Harikrishnan, 2005]. Williams et al. [2011] examined the relationship between the TSR value and the amount of Fractionated Reclaimed Asphalt Pavement (FRAP). They found no significant correlation, but they found most of the mixtures evaluated exceeded a minimum TSR of 80%.

Some authors have studied options to improve the moisture resistance of mixtures by using anti-stripping agents. Khosla and Harikrishnan [2005] found the use of hydrated lime and anti-stripping agents did not produce an appreciable reduction of indirect tensile strength and TSR values. Finally, the authors also found the fatigue life decreases exponentially with the reduction of the indirect tensile strength, which can be due to a loss of stiffness and the origination of cracks and stripping in the pavement. This study suggested there is a minimum indirect tensile strength for a particular ESAL range [Khosla and Harikrishnan, 2005]. Liang [2008] subsequently found aggregate source, method of compaction, specimen size, loose mix aging, freeze-thaw conditioning, and saturation level can change TSR values.

Liang [2008] aimed to assess the applicability of this method for Superpave mix design of Hot Mix Asphalt (HMA) in Ohio. Liang [2008] recommended the following adjustments to the AASHTO T283 test method for Superpave mix designs: a 150-mm gyratory specimen should be used (since TSR was strongly correlated to specimen dimensions); only 4 hours of conditioning was needed; just 24 hours of compacted HMA aging was necessary (as it was found aging does not significantly affect TSR values); one freeze-thaw cycle should be included; and the saturation level should increase to 80-90%.

Moaveni and Abuawad [2012] compared two approaches or methods to determine the stripping resistance of asphalt mixture, one was the AASHTO T-283 and the other one was a modification developed by the Illinois Department of Transportation (IDOT). The two methods only differ in the conditioning procedure: The AASHTO method considers the curing time during mixing and the freezing time, which the IDOT method does not. The TSR values indicated the IDOT method underestimates the effect of moisture damage. In fact, the fracture energy between dry and conditioned samples differed more significantly in the AASHTO T-283 than in the IDOT method. Hence, researchers concluded freeze-thaw cycle must be included in any stripping resistance analysis of a mixture [Moaveni and Abuawad, 2012].

Moaveni and Abuawad [2012] also suggested the fracture resistance of a mixture can be considered as an indicator of the moisture susceptibility, but they recommended more laboratory testing on various types of mixtures using different methods of conditioning to better clarify the effect of sample conditioning on mixture fracture energy. On the other hand, West et al. [2013] evaluated the effect of using high RAP content (between 25 and 55%) on moisture damage and found that even if a mix with high RAP content developed lower fracture energy, its susceptibility to moisture damage was similar to the control mix with no RAP.

In-Place Air Voids

Dating back several decades, researchers understood the importance on in-place air voids (density), with Lee [1969] stating, "a longer life is associated with higher asphalt content and increased density." And later, Hughes [1989] would state "compaction is the single most important factor that affects pavement performance in terms of durability, fatigue life, resistance to deformation, strength, and moisture damage". Echoing this sentiment, Brown [1990] stated "density is one of the most important parameters in construction of asphalt mixtures." To this day, the general consensus remains, in-place air voids is one of the most important parameters affecting AC performance [Bonaquist, 2014].

As noted by Hughes [1989], in-place air voids effects many important parameters related to performance. In general, high air voids are associated with higher permeability allowing for water and air to enter the pavement which can result in water damage, oxidation, raveling, and cracking, while "low air voids lead to rutting and shoving" [Brown, 1990]. Alderson [2014] reported similar effects, summarizing the effect of decreased in-place air voids as follows:

- Reduced rate of oxidative age hardening
- Increased mix stiffness
- Reduced rutting potential, except at very low air void content Increased resistance to fatigue
- Decreased permeability
- Increased resistance to moisture damage

Previous research has associated the degree of compaction, as measured by in-place air voids, with performance. Brown [1990] indicated the initial in-place air voids should be 8% at the greatest and no less than 3% on the lowest end. Generally, for each 1% increase in air voids beyond 7% there is about a 10% loss in pavement life [Linden et al., 1989]. More specifically, Tran et al. [2016] reported "a 1% decrease in air voids was estimated to improve the fatigue performance of asphalt pavements between 8.2 and 43.8%, to improve the rutting resistance by 7.3 to 66.3%, and to extend the service life by conservatively 10%."

There are many factors that affect pavement density, or in-place air voids. Hughes [1989] focused on material properties including aggregate properties, asphalt binder properties, and mix properties. Hughes [1989] identified several aggregate properties that play an important role in achieving density: particle shape or angularity, absorption, and surface texture. Hughes [1989] also identified gradation as influenced by maximum aggregate size, concentration of coarse aggregate, amount of sand sized material, and the amount of filler, as important aggregate properties. Additionally, Hughes [1989] identified the following asphalt binder and mix properties: asphalt binder viscosity, asphalt content, filler to asphalt ratio, compaction temperature, and the "fluid content" (the sum of the asphalt content and moisture content in the mix).

In the NCHRP Report 531 [Brown et al., 2004] the relationship between air voids, lift thickness and permeability was investigated. It was reported that coarse-graded mixtures are generally more permeable than fine-graded mixes for a given air void level. Brown et al. [2004] also identified an important ratio, the lift thickness to nominal maximum aggregate size (NMAS) ratio or t/NMAS, recommending for improved compactibility, t/NMAS should be a minimum of 3 for fine graded mixes and 4 for coarse graded mixes. This t/NMAS was later included in "A Manual for Design of Hot Mix Asphalt with Commentary" in which minimum and maximum ratios were identified for various mix types as shown in Table 11-2. Based on the t/NMAS ratio recommendation, and their recommended NMAS for each course, the authors recommended minimum and maximum lift thicknesses; the recommended lift thicknesses for the base course is shown in Table 11-3.

Table 11–2 Recommended t/NMAS ratios [Advanced Asphalt Technologies, 2011].

Mixture Type	Minimum t/NMAS	Maximum t/NMAS
Fine, Dense-Graded	3.0	5.0
Coarse, Dense-Graded	4.0	5.0
Gap Graded HMA	4.0	5.0

Table 11–3 Recommended AC base lift thicknesses Advanced [adapted from Asphalt Technologies, 2011].

	Recommended Lift Thickness, mm (in)	
NMAS, mm (in)	Fine-graded mixtures	Coarse-graded mixtures
19.0 (0.75)	60 (2.5) to 100 (4)	75 (3) to 100 (4)
25.0 (1.0)	75 (3) to 125 (5)	100 (4) to 125 (5)
37.5 (1.5)	115 (4.5) to 150 (6)	150 (6)

As part of a workshop conducted through a recent cooperative initiative between Federal Highway Administration (FHWA) and Asphalt Institute (AI) on “Enhanced Durability Through Increased In-Place Pavement Density,” factors affecting in-place air voids were provided [FHWA and AI, 2016]. While Hughes [1989] focused on material properties, FHWA and AI [2016] focused on construction parameters that can influence density. The factors identified by FHWA and AI [2016] are listed below.

- Base condition
- Lift thickness vs. NMAS
- Laydown temperature
- Ambient conditions
- Cooling rates
- Balancing production through compaction
- Paver operations

Appendix B: Specification Review

Table 11–4 Summary of state specifications

	States w/ asphalt base requirements							States w/o specific base requirements							
Specification Item	OH	MI	IN	PA	SC	VA	TX	KY	IL	ON	WI	MD	CA	CO	KS
Mixture Related Requirements															
Gradation	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Binder content	Yes	Yes	Yes	Yes	Yes	Yes		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Binder type/grade	Yes	No	Yes	Yes	Yes	Yes			Yes				Yes		
VMA	Some	Yes	Yes	Yes		Yes	Yes		Yes		Yes		Yes		Yes
RAP allowed	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes			Yes			Yes
RAP minimum or maximum	Yes	No	Yes	Yes			Yes	Yes				Yes			Yes
Pavement Layer Related Requirements															
Layer placement depth specifics	Yes	Yes	Yes	Yes	No	No	Yes	Yes	Yes		Yes				Yes
Placement temperature	Yes	Yes	Yes	Yes	Yes	Yes						Yes			
Aggregate Requirements															
Aggregate requirements	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes			Yes		Yes	Yes	Yes
Physical property requirements	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes			Yes		Yes		Yes
Specific Aggregate Tests															
Percent of wear, Los Angeles test, maximum	Yes	Yes	Yes		Yes						Yes		Yes		
Unit weight, compacted, minimum (slag)	Yes														
Loss, sodium sulfate soundness test, max	Yes		Yes		Yes			Yes			Yes				
Percent by weight of fractured pieces, min	Yes				Yes						Yes		Yes		
Micro-Deval abrasion loss test, max (coarse aggregate gravel only)	Yes														
Fine aggregate angularity		Yes	Yes								Yes		Yes		Yes
% Flat and elongated particles		Yes									Yes		Yes		
% soft particles (max)		Yes													
% sand equivalent (min)		Yes									Yes		Yes		Yes
Freeze and thaw			Yes								Yes				
Brine freeze-thaw			Yes												
Coarse aggregate angularity			Yes										Yes		Yes
Clay content			Yes												
Local sand allowed					Yes										
Limestone allowed? (CA/Screenings)					Yes										
Absorption, max %					Yes										
Crusher run/asphalt sand allowed?					Yes										
Acceptance and Testing Related Requirements															
Detailed acceptance criteria		Yes	Yes	Yes	Some	Yes	Yes		Some						
Tolerances specified	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes						
Types of acceptance tests															
Density			Yes	Yes	Yes		Yes	Yes	Yes	Yes	Yes	Yes	Yes		Yes
Air voids			Yes				Yes	Yes	Yes		Yes	Yes	Yes		Yes
Binder content	Yes		Yes		Yes	Yes	Yes	Yes	Yes		Yes	Yes	Yes		Yes
Aggregate gradation	Some		Yes				Yes	Yes	Yes		Yes	Yes	Yes		Yes
Segregation		Yes					Yes								
Rutting		Yes													
Flushing		Yes													
Edge of paved shoulder		Yes													
Crack		Yes													
Discharge temperature			Yes			Yes									
VTM						Yes						Yes			
VMA						Yes			Yes		Yes	Yes	Yes		Yes
VFA						Yes			Yes		Yes	Yes			Yes
F/A						Yes									
Hamburg wheel test							Yes						Yes		
Smoothness													Yes		
Boil test							Yes								

Appendix C: District Survey

Ohio University is conducting an ODOT sponsored research project to evaluate the construction and acceptance criteria for asphalt base (301, 302, 880). The research team will be collecting cores from a sample of pavements constructed since the early 1990's. The team would appreciate your response to the questions below:

- If project(s) from your district are selected for sampling, who (name, phone number, and email) can we contact to obtain construction records? Maintenance of traffic?

  Construction records: ______________________________

  MOT: ______________________________

- The research team will collect samples from ongoing projects to serve as a control. Please list projects where 301 or 302 asphalt base will be place early in 2017

One objective of the research is to evaluate the adequacy of ODOT's current practice. As part of this evaluation, the research team would like to evaluate any flexible pavement which met ODOT acceptance criteria but has performed poorly. If you have a flexible pavement section constructed in your district since 1990, which have performed poorly, please answer the following questions:

1. Please provide the county-route-section and project number for each poorly performing project

2. For each project listed for question 2, please provide the mode of failure, i.e. rutting, wheel track cracking, transverse cracking, roughness, potholes, etc.

3. For each projects listed for questions 2, was the cause of failure investigated? If yes, please provide a copy or a summary of the investigation.

Please email your response to greenr1@ohio.edu by October 28, 2016. If you have any questions, please contact Roger Green at 614-519-6153.

Table 11–5. Summary of ODOT District survey responses.

ODOT District	Premature Failure			Active projects		
	PID	County-Route-Section	comments	PID	County-Route-Section	comments
1	none			none		
3	1032-93	MED 76 7.00-12.03	Failing base has been removed and replaced. Asphalt binder came off the aggregate.	92954	MED US 0042 17.80	Section improvement
	428-94	MED 76 0.00-7.00	Failing base has been removed and replaced. Asphalt binder came off the aggregate.	98078	RIC CR 0281 00.58 (Trimble Rd.)	Major widening
	8002-90	ASD 250 12.75-14.97	New bypass	97441	LOR CR 0032 01.98 (Middle Ridge)	Intersection improvement
	33-02	CRA/RIC 30 20.50-	New bypass – 880 Warranty	97611	ERI SR 0013 02.50 GSM	Slide repair
	36-02	CRA 30 14.91-20.50 *	New bypass – 880 Warranty	93868	D03 Mohican SP FY2016	Slide repair
	509-02	CRA 30 9.86-15.00 *	New bypass – 880 Warranty	98655	HUR CR 0705 00.00 (Milan Ave)	Major rehabilitation
	3011-11	MRW 71 12.19-19.54	Third lane widening	95089	WAY East Pine Street	New construction
				96257	HUR US 0224 07.60	Drainage system maintenance/repair (NEW)
				99546	ASD CULVERT FY2017 (B)	Culvert construction/reconstr/repair
				97439	LOR CR 0051 07.77 (Baumhart)	Intersection improvement
				104003	LOR SR 0010C 00.50	Intersection
				99548	CRA SR 0103 16.89	Culvert construction/reconstr/repair
		* subgrade failure		99549	RIC CULVERT FY2017	Culvert construction/reconstr/repair
5	16293	FAI-33-0.78	Pavement coring completed October 7, 2014	86245	LIC-13D-0.00	
	16294	FAI-33-13.21	All of the cores extracted showed some level of deterioration (stripping) in the asphalt base	87935	LIC-310-0.74	
				97879	LIC-161-1.83	
6		FAY-35-2.57		25594	FRA-60-3.41	
				84868	FRA-71-5.29	
				76469	FRA-270-9.15	
				81747	FRA-270-21.67	
7	77248	MOT-75-6.36	Rutting on Dixie Drive at intersections	Small quantities to rebuild approaches at bridge projects		
9	none			92080	PIK-CR50-0.46	Bridge replacement
				101463	JAC-93-16.80	New urban pavement
				84964	SCI-139-1.66	Bridge replacement
10	none			84468	ATH-33-11.74	
11	none			80599	BEL-70-14.24	Interchange reconstruction
				95916	CAR-9-17.63	Intersection Improvement

Appendix D: Sampling and Testing Plan

The sampling and testing plan approved by the technical advisory committee (TAC) follows. The final matrix of projects shown in Chapter 5 differ from the matrix below due to the inability to access some sites because of coring restrictions in some urban areas, active construction, change in material type, etc.

The goal of this research was to improve ODOT's current asphalt base acceptance methods, which will improve the quality of asphalt base pavement construction. Common types of asphalt pavement distress include cracking due to fatigue, temperature cycles, or aging; disintegration due to moisture and/or freeze/thaw; and rutting due to repeated loading and/or weak foundation. Due to the traffic volumes typical of Ohio's two lane system, designed asphalt pavements in Ohio are typically greater than 9 in (229 mm) thick. This thick pavement design results in low vertical strains on the asphalt base layer, and rutting of the base has not been a problem on state routes in Ohio. Therefore, rutting was not a consideration in developing the sampling and testing plan.

Air voids play a vital role in the design of asphalt concrete (AC). When the air voids are too low, the compacted mix will often bleed, resulting in a loss of strength and surface friction. When the air voids are too high, oxidation occurs more rapidly, resulting in durability issues. High air voids also create higher permeability. The increased presence of water can lead to stripping the binder from the aggregate, which also creates a loss of strength and durability. The bulk density achieved in the field is a parameter important to predicting performance of the mix. Often, the maximum specific gravity of the mix (G_{MM}) is not determined for Item 301 AC base mixes in Ohio. The G_{MM} measured as part of the mix design process can potentially be significantly different than the G_{MM} measured from the plant produced mix, therefore the G_{MM} must be determined using the field cores collected from the sites.

Moisture induced stripping in the mix can be detrimental to the AC pavement, and measuring the moisture susceptibility is vital to understanding performance. The effects of the resulting loss of strength can either be sudden or prolonged over many years; either way, it can be fatal for a pavement system, especially in the base layers where bottom-up cracking is the controlling failure mode for the pavement structure. AASHTO T 283 directly evaluates the effects of moisture damage by introducing an asphalt specimen to an accelerated saturation process combined with a freeze-thaw cycle. This method, widely accepted by researchers and DOTs, measures and compares the non-conditioned (dry) Indirect Tensile Strength (ITS) to the conditioned (wet) ITS of field compacted core samples in the form of the Tensile Strength Ratio (TSR). ODOT requires the TSR be greater than 0.80 for Item 442 mixes and greater than 0.70 for all other surface and intermediate mixes. Currently, AC Base mixes do not have a TSR limit. However, evaluating TSR of the in-place AC base mixes would help identify if the mixes are susceptible to moisture damage. .

The standard method of test for determining the fracture potential of asphalt mixtures using the Semicircular Bending Geometry (SCB) at intermediate temperature, AASHTO TP 124, was selected to test the cracking potential of the AC base specimens. This method was introduced by Al-Qadi et al. [2015] in the Report FHWA-ICT-15-017 as an improved version of the AASHTO TP 105 method. The previous version had shown potential as a QA/QC test of fracture properties using SCB [Saadeh and Eljairi, 2011]. The SCB test has been used in different studies as a method to characterize low-temperature cracking potential in AC mixtures [Wagoner et al., 2005]. Including studies on mixtures with different percentage of reclaimed asphalt pavement (RAP) and reclaimed asphalt shingles (RAS) such as Li et al. [2008], Williams (2010), and Behnia et al. [2011].

Marasteanu et al. [2007] have found the fracture toughness and energy obtained from testing protocols such as SCB or disc compact tension (DCT) test correlated best with the field distresses measured in pavement sections than other methods such as direct tension test (DTT) or indirect tensile test (IDT). In later research, Marasteanu et al. [2012] found an agreement existed in the fracture energy prediction showed by the SCB and DCT test. Although both tests have shown to be an appropriate option to measure the low-temperature cracking resistance of AC mixtures, the DCT is, overall, a more complex test and has not been completely evaluated in its ability to identify differences in mixes and its sensitivity to asphalt content and temperature variations [Walubita et al., 2013].

Al-Qadi et al. [2015] used several tests, such as low-temperature SCB (AASHTO TP 105), low-temperature DCT, IDT, dynamic modulus test, Texas overlay test, and push-pull fatigue test, to evaluate properties and performance of different AC mixtures. None of these conventional tests were found appropriate to accurately and consistently predict and rank the cracking resistance of AC mixes [Al-Qadi et al., 2015].

Consistency or repeatability has already been considered the weak aspect of the SCB (AASHTO TP 105) test [Walubita et al., 2013]. After Al-Qadi et al. [2015] evaluated their results and considered the SCB test's potential low-cost implementation, its equipment availability, and the relatively easy specimen preparation and testing, they decided to improve the SCB method by introducing the Flexibility Index (FI). The FI made the SCB test better able to capture the effect of changes in mixes than captured by fracture energy alone. The improved method to evaluate fracture potential and flexibility index were validated using plant-produced, laboratory-produced, and field core specimens to predict the cracking resistance among mixes. It was further validated with finite element simulations [Al-Qadi et al., 2015].

From the AASHTO TP 124 protocol, it is possible to obtain fracture energy (G_f), peak load (P_{max}), slope at the post-peak inflection point (m), and the flexibility index (FI). The FI captures cracking resistance and describes the fundamental fracture processes consistent with the size of the crack tip process zone. Hence, the FI provides a means to identify brittle mixes that are prone to premature cracking [Al-Qadi et al., 2015]. The AASHTO TP 124 is an improved and more consistent method for determining the fracture potential of asphalt mixtures using the Semicircular Bending Geometry (SCB) at intermediate temperature that should be appropriate for this project.

Quantifying durability due to construction and mix composition for dense-graded AC mixes has been a debatable topic. The Cantabro Loss test has long been used in the design of open-graded friction course (OGFC) mixes to ensure durable mixes in Europe and South Africa [Huber, 2000]. It has gained traction in the United States, becoming more prevalent in evaluating durability of porous mixes in the mix design phase. Recent studies have shown the Cantabro Loss test can also be used to evaluate durability of dense graded mixes [Doyle and Howard, 2014]. For dense graded mixes, it has been reported the test is sensitive to mix design properties such as air voids, binder content, and RAP content [Doyle and Howard, 2014]. The Cantabro Loss test involves placing a cylindrical AC specimen in the Los Angeles Abrasion machine and subjecting the specimen to 300 revolutions at 30 to 33 revolutions per minute at room temperature without the use of steel balls, per test procedure Tex-245-F. The weight of the specimen is measured before and after the test. The Cantabro Loss in percent is determined by dividing the amount of material lost after being subjected to the test by the initial weight of the specimen. The test is very easy to run and takes little time to complete. The Texas DOT has a standard for measuring the Cantabro Loss (Tex-245-F). Currently, there is no standard criterion for the Cantabro Loss for a dense-graded AC mix, but the nature of the test could provide insight into the durability of dense-graded AC. Based on the use of Cantabro testing in identifying durable OGFC mixes and the promising findings when applied to dense graded mixes, it is proposed the Cantabro Loss tests be conducted to further evaluate asphalt base mix at locations which failed by TSR. Additionally, a sampling of the locations which passed TSR and for which additional cores can be obtained during the scheduled maintenance of traffic, will be selected for Cantabro loss test.

Therefore, the tests proposed for Phase I of this project include:

1. AASHTO T 166 – Standard Method Of Test For Bulk Specific Gravity (G_{MB}) Of Compacted Hot Mix Asphalt (HMA) Using Saturated Surface-Dry Specimens. Measures will be taken to ensure cores will not be cured.

2. AASHTO T 209 – Standard Method Of Test For Theoretical Maximum Specific Gravity And Density Of Hot-Mix Asphalt. Samples will not be cured at elevated temperatures. Tests will not be performed while samples are warm. The SSD procedure, per ODOT Supplement 1036.01 D 2, will be used for cores.

3. AASHTO T 283 – Standard Method Of Test For Resistance Of Compacted Asphalt Mixtures To Moisture-Induced Damage.

4. AASHTO TP 124 – Standard Method Of Test For Determining The Fracture Potential Of Asphalt Mixtures Using Semicircular Bend Geometry (SCB) At Intermediate Temperature.

5. TEX-245-F – Cantabro Mass Loss.

A minimum of fourteen 6 in (152 mm) diameter cores will be collected at each site to provide an adequate number of specimens to conduct the above tests, as described below. Cores will be collected at random locations along a 1000 ft (305 m) lane length using the procedure on ODOT Form TE-217. Only the top lift of asphalt base will be used for evaluation.

- Bulk Specific Gravity (G_{MB}) will be measured on all AC base specimens.
- One Theoretical Maximum Specific Gravity (G_{MM}) measurement per site will be performed. AASHTO 209 requires 4000 g (8.2 lb) of mix for maximum aggregate sizes of 1.5 in (38 mm). Depending on the thickness of the top layer, more than two cores may need to be devoted for G_{MM} in order to have enough material for the test.
- Six specimens, three dry and three conditioned, will be prepared to evaluate the moisture susceptibility by determining the tensile strength ratio (TSR). Dry and wet ITS strength values will also be reported.
- For the SCB test, three specimens will be used.
- Two specimens will be needed for the Cantabro Loss test.
- Two additional specimens will be collected for use should a specimen be damaged during preparation.

A review of construction records and a survey of ODOT District personnel identified 58 projects which include Item 301 and Item 302 asphalt base with and without RAP, and with a range of ages. Projects with poor performance which may be due to asphalt base quality were also identified. The 50 projects evaluated were selected from this list, with a few substitutions when required. The final list of the 50 projects evaluated is in Table 11–6 and Table 11–7. Due to the limited number of projects in some areas of the state, an equal geographic distribution of projects between the five regions of the state (northeast (NE), northwest (NW), center (CEN), southeast (SE), southwest (SW)) could not be achieved.

Table 11–6. Asphalt base acceptance project list.

Spec. Year(s)	Region	C-R-S	Project No	PID	AC Base Item	Notes
1995 through 2002 (design air = 5.0%)	NW	LUC-2-21.24	990141	9159	301	
	NW	WOO-795-2.01	990505	13725	302	
	NE	TUS-800-19.54	990153	13321	301	
	NE	WAY-30-11.86	B312539	040044	301	
	NE	CUY-71-0.00	980748	15717	302	
	NE	WAY-30-11.86	B322255/ B322228	040044	302	
	CEN	FAY-35-0/.00	000091	4388	301	
	CEN	COS-36-21.59	960278	14142	302	
	SW	WAR-71-3.78*	990780	10696	302	
	SE	VIN-50-11.75	010123	10504	301	
	SE	WAS-7-22.99	980099	14564	302	One Lane Only
2005+ specs, Item 880 w/ RAP up to 30% RAP if virgin binder >3.4%. design air = 4.5% (2002 to 2008)	NW	DEF-24-7.96	060087	24337	301	
	NW	HAN-30-3.00	050003	77302	302	
	NE	MED-71-9.56	050343	14018	301	
	NE	SUM-77-21.79	060151	16514	302	
	CEN	FRA-161-23.20	060150	24486	301	
	SW	CLI-73-12.03	060413	78569	301	
	SW	CLI-73-6.52	090244	78571	302	
2008+ specs. w/ < 5% RAP design air = 4.0 %	NW	MER-219-14.04	050313	19968	301	no RAP
	NW	FUL-20-10.86	000341	19342	302	no RAP
	NE	WAY-30-17.40	010199	10289	301	no RAP
	NE	ASD-39-0.00	040500	23578	302	no RAP
	NE	LAK-2-7.76	79545	100215	302	
	CEN	MAD-142-0.49	010317	11739	301	no RAP
	CEN	DEL-23-19.24	970335	16350	302	no RAP
	CEN	MRW-71-3.17	133001	86920	302	
	SW	ROS-207-0.00	040533	18492	301	no RAP
	SW	HAM-264-6.87	000135	13853	302	no RAP
	SW	CLA-70-13.98	100243	84664	302	
	SW	BUT-75-5.91	080246	75971	302	
	SE	GAL-35-8.32	070334		301	no RAP

Table 11–7. Asphalt base acceptance project list. (ctd)

Spec. Year(s)	Region	C-R-S	Project No	PID	AC Base Item	Notes
Poor Performance	NE	ASD-250-12.75	908022		302	
	NE	CRA-30-9.864	20509		302	880 specification, District attributes failure to A-4a soil
	CEN	FAI-33-13.21	020110	16294	301	
	CEN	FAI-33-7.81	010136	16293	301	
	CEN	FAY-35-2.57	000577		302	2.60 to 4.52 failed, was repaired and overlaid, need rehab plan also
	CEN	MRW-71-12.19	1103011		302	Asphalt base failed, replaced, district has cores
	SW	CLI-73-6.525	090244	78571	302	880 mainline, 301 side roads
Control	NW	HAN, WOO, LUC-75		projects (5)	302	
	NW	LUC-75-6.70	140536	76032	302	
	NW	HAN/WOO-75-19.22/0.00	143000	95437	302	
	NW	WOO-75-19.43	140237	25521	302	
	NW	LUC-75-4.52	140485	77254	302	
	NW	WOO-75-10.61	140170	95435	302	
	NE	MED-42-17.80	160430	92954	301	
	NE	CR 0281		98078	301	
	NE	CUY-77-0.00	163019	79671	302	
	CEN	LIC-13D-0.00	168006	86245	301	
	CEN	LIC-161-1.83	168030	97879	301	
	CEN	LIC-310-0.74	160043	87935	301	
	CEN	FRA-71-5.29	150395	84868	302	
	CEN	FRA-70-3.41	150396	25594	302	
	CEN	FRA-270-21.67	150249	81747	302	
	SW	WAR-71-14.45	100280	22950	302	
	SE	JAC-93-16.80		101463	301	
	SE	BEL-70-14.24		80599	301	
	SE	CAR-9-17.63		95916	301	
	SE	ATH-33-11.74		84468	302	
	SE	JAC-93-16.80		101463	302	

Appendix E: Traffic and Performance Data

ODOT has adopted the AASHTO Guide for Design of Pavement Structures, hereafter referred to as the AASHTO Guide, for pavement thickness design. (http://www.dot.state.oh.us/Divisions/Engineering/Pavement/Pavement%20Design%20%20Rehabilitation%20Manual/Complete_PDM_2016-07-15_version.pdf). Evaluation of the thickness design procedure was not included in the scope of this project given its widespread use. However, aspects of the design procedure affected by the construction process and damage to the asphalt base, which could affect the results of the laboratory testing, were considered.

The Modeling and Forecasting Office at ODOT provides the design engineer "current" traffic, design year traffic and percent trucks which are used in the thickness design process. The "current" and 20 year design truck traffic, and percent trucks for projects included in this study were collected from construction plans. This information is typically shown on the title sheet or the plan and profile sheets in the construction plans, such as the example in Figure 11-1.

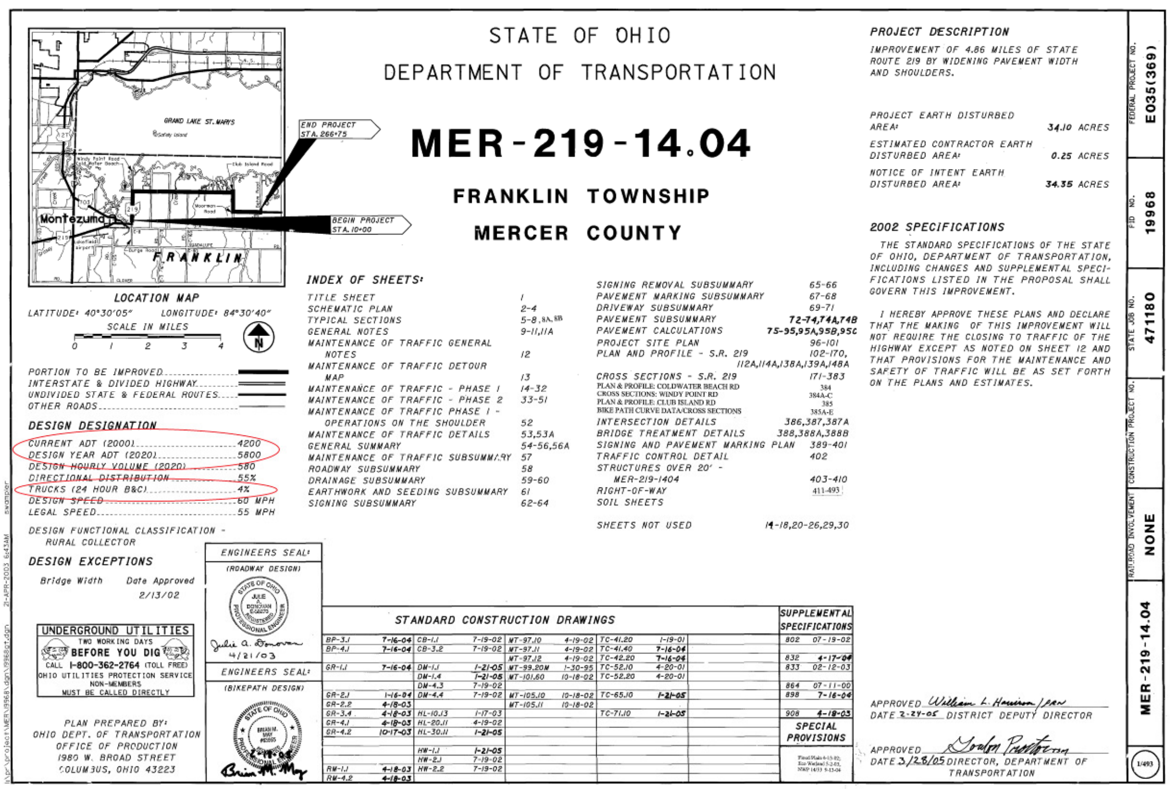


Figure 11-1. Construction plan for MER-219.

The most recent traffic count for each section was obtained from ODOT's Office of Technical Services website, http://odot.ms2soft.com/tcds/tsearch.asp?loc=Odot&mod= (see Figure 11-2 and Table 11–8). The construction traffic information was used to calculate the predicted truck traffic, assuming linear growth, corresponding to the year of the most current traffic count. Sections constructed after 2012 were not included in this analysis due to a lack of traffic reports.

The results are presented in Figure 11-3. As shown in the figure, eight of the sections in this study had a ratio of measured truck traffic to predicted truck traffic slightly or significantly greater than 1.0. Of these 8 sections, one, VIN-50, were overlaid 10 years after construction. A rule of thumb, when using the AASHTO pavement design equations, is each additional inch (25 mm) of asphalt will double the fatigue life of the pavement [http://www.apao.org/asphalt_thinlay.html]. Based on this rule of thumb, 59% of the sections were within an inch (25 mm) of the thickness needed for the measured traffic. 16% of the sections are estimated to have fatigue life reduced by 50% or more due to actual truck traffic exceeding the estimated value.

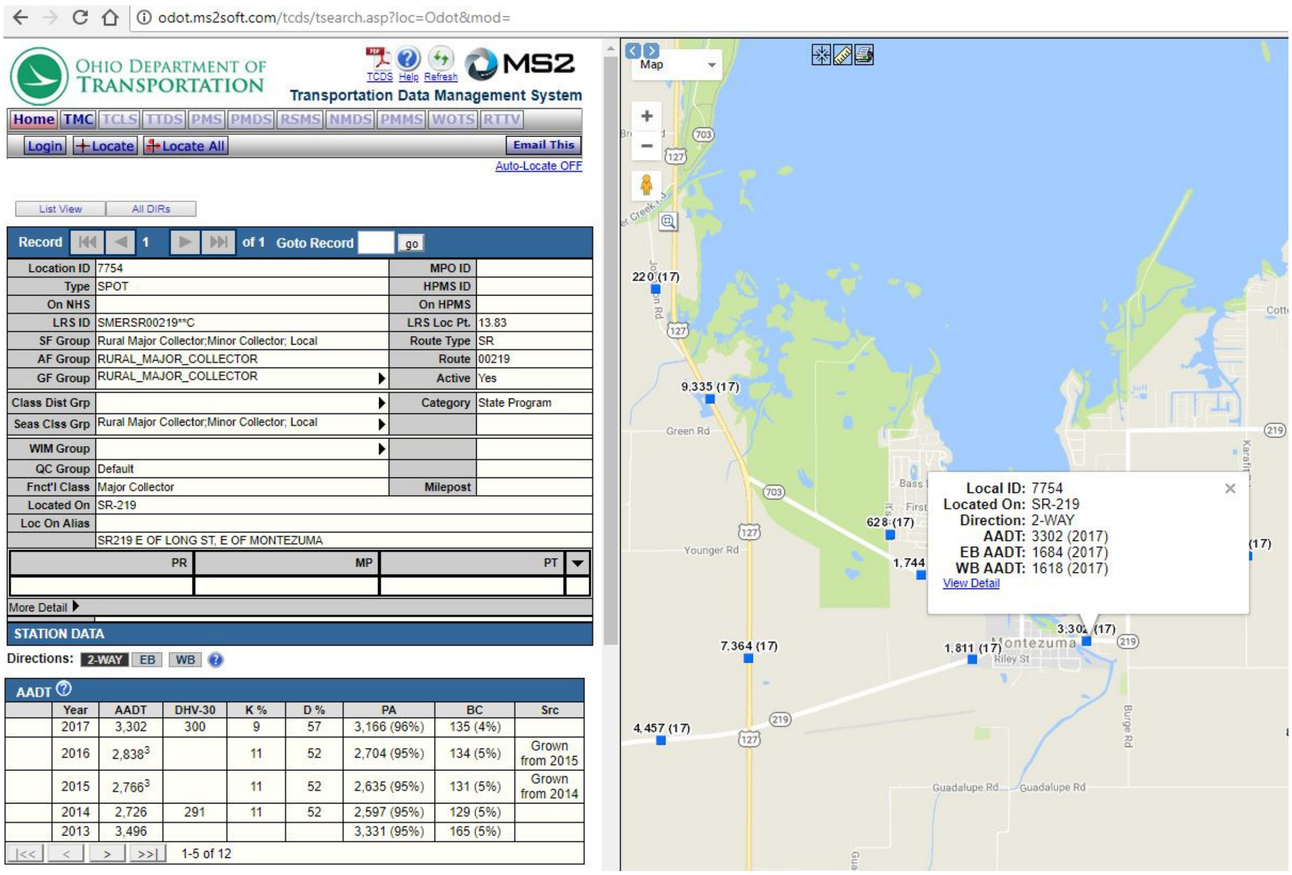


Figure 11-2. Mercer County traffic counts as displayed on ODOT Technical Services web site. [http://odot.ms2soft.com/tcds/tsearch.asp?loc=Odot&mod=]

Table 11–8. Measured and predicted truck traffic.

							Design Traffic					Measured Traffic						Measured/
County	Route	Section	Project Number	Begin SLM	End SLM	Completion date	"Current" year	"Current" year ADT	Design year	Design year ADT	% Trucks	Year	Age (yr)	Measured ADT	Predicted ADT	Measured ADTT	Predicted ADTT	Predicted ADTT
WOO	795	2.01	1999-0505	4.87	5.13	2000	1999	16660	2011	22285	9%	2017	17	12573	25098	314	2259	14%
CLI	73	12.03	2006-0413	12.41	12.63	2008	2010	22320	2030	28140	10%	2017	9	6648	24357	599	2436	25%
CLI	73	6.52	2009-0244	7.00	7.18	2011	2010	14010	2030	21230	20%	2017	6	8564	16537	976	3307	30%
HAM	264	6.87	2000-0135	6.95	7.07	2000	1995	48802	2015	88182	1%	2017	17	16734	92120	295	921	32%
ROS	207	0.00	2004-0533	17.00 on SR 104	17.18	2006	2008	9800	2028	14600	15%	2017	11	12816	11960	633	1794	35%
LUC	2	21.24	1999-0141	23.67	23.94	2001	1999	32720	2019	44490	8%	2017	16	20419	43313	1274	3465	37%
WAR	71	14.20	2010-0280	15.60	15.87	2015	2009	49820	2029	70190	37%	2017	2	97568	57968	8533	21448	40%
LAK	2	7.60	2010-0215	11.93	12.34	2012	2010	55100	2030	66800	5%	2017	5	52795	59195	1479	2960	50%
SUM	77	13.54 (21.79)	2006-0151	19.07	19.34	2008	2009	85830	2029	102500	13%	2017	9	86263	92498	6038	12025	50%
MRW	71	3.17	2013-3001	5.40	5.57	2015	2012	52870	2032	67800	38%	2017	2	53170	56603	11692	21509	54%
MAD	142	0.49	2001-0317	1.82	2.08	2003	1996	5282	2016	6716	4.2%	2017	14	3101	6788	155	285	54%
COS	36	20.83	1996-0278	23.67	23.85	1998	1995	10400	2015	13510	23%	2017	19	8439	13821	1773	3179	56%
MER	219	8.72 (14.04)	2005-0313	14.32	14.51	2005	2000	4200	2020	5800	4%	2017	12	3302	5560	135	222	61%
FUL	20	6.75 (10.86)	2000-0341	19.98	20.06	2000	1998	4770	2017	6424	32%	2017	17	5458	6424	1323	2056	64%
MED	71	5.94 (9.56)	2005-0343	12.31	12.58	2007	2005	36700	2025	58330	26%	2017	10	43328	49678	9021	12916	70%
FAI	33	13.25	2002-0110	16.21	16.47	2003	2001	21550	2021	29210	9%	2017	14	23417	27678	1748	2491	70%
BUT	75	5.91	2008-0246	ramp		2010	2002	87180	2030	119310	22%	2017	7	136575	104393	17619	22966	77%
DEL	23	19.24	1997-0335	19.26	19.52	1997	1997	25820	2017	38420	20%	2017	20	27703	38420	5928	7684	77%
DEF	24	4.94 (7.96)	2006-0087	8.29	8.51	2008	2008	9590	2028	12710	46%	2017	9	11023	10994	4005	5057	79%
LIC	161	14.41 (23.2)	2006-0150	2.37	2.63	2008	2010	37000	2030	59100	5%	2017	9	36631	44735	1831	2237	82%
WAY	30	11.86	2004-0044	14.92	15.49	2006	2006	17720	2026	23180	21%	2017	11	21626	20723	3775	4352	87%
CLA	70	13.98	2010-0243	17.99	18.16	2012	2012	50800	2032	60500	36%	2017	5	55594	53225	16927	19161	88%
FAI	33	7.31	2001-0136	9.41	9.60	2003	2001	18970	2021	25530	8%	2017	14	23417	24218	1748	1937	90%
CUY	71	0.00	1998-0748	1.12	1.38	1999	1995	76410	2020	105570	6%	2017	18	63604	102071	5725	6124	93%
HAN/WYA	30	3.00	2005-0003	5.04	5.30	2007	2007	6160	2027	8530	56%	2017	10	9248	7345	4545	4113	110%
ASD	39	0.00	2004-0500	0.42	0.68	2005	2004	2460	2016	2750	10%	2017	12	2256	2774	366	277	132%
VIN	50	11.75	2001-0123	12.17	12.37	2001	1999	2853	2019	3672	10%	2017	16	6202	3590	637	359	177%
WAR	71	3.78	1999-0780	9.66	9.77	2001	1997	23490	2017	36490	17%	2017	16	66957	36490	13712	6203	221%
FAI	33	19.79	2003-0046	19.92	20.11	2005	2001	0	2021	25480	5%	2017	12	16997	20384	2407	1019	236%
FAI	33	17.44	2002-0446	19.26	19.45	2004	2001	0	2021	25480	5%	2017	13	16997	20384	2407	1019	236%
GRE/FAY	35	26.20	2000-0091	26.71	26.90	2001	1997	6720	2017	8350	17%	2017	16	11456	8350	3529	1420	249%
FAY	35	2.57	2000-0577	5.21	5.47	2001	1997	3430	2017	4410	12%	2017	16	14090	4410	4946	529	935%

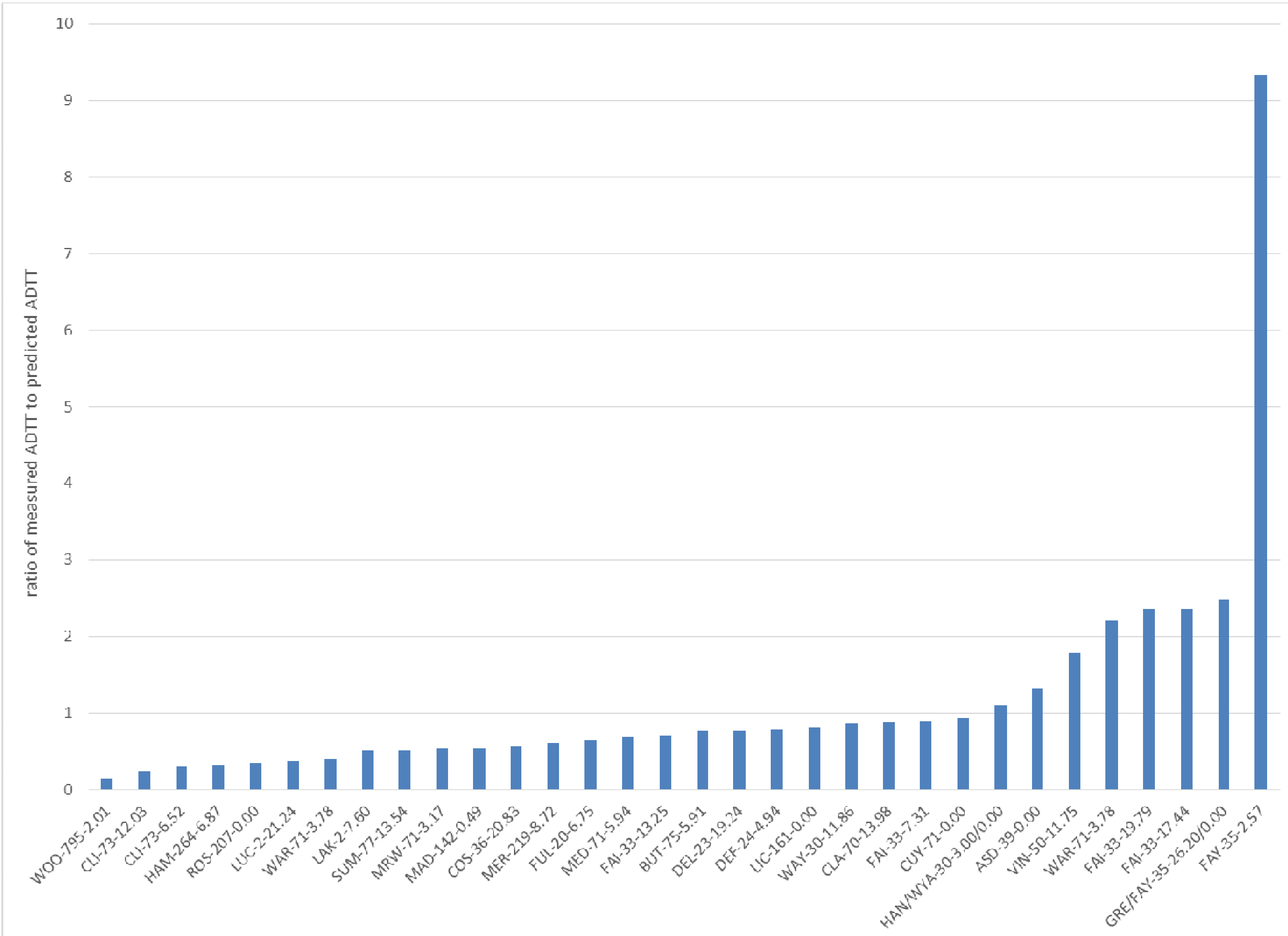


Figure 11-3. Ratio of measured ADTT to predicted ADTT for projects built before 2012.

ODOT tracks pavement performance using Pavement Condition Rating (PCR), which is a visual rating of pavement distresses [https://www.dot.state.oh.us/Divisions/Planning/TechServ/TIM/Documents/PCRManual/2006PCRManual.pdf]. The rating form shown in Figure 11-4 is used to record ratings of distresses for a flexible pavement.

Section: ____________________
Log mile: ______ to__________
Sta: __________ to__________

FLEXIBLE

Date: ____________________
Rated by: ________________
# of Utility Cuts ____________

PAVEMENT CONDITION RATING FORM

DISTRESS	DISTRESS WEIGHT	SEVERITY WT.*			EXTENT WT.**			DEDUCT POINTS***
		L	M	H	O	F	E	
RAVELING	10	0.3	0.6	1	0.5	0.8	1	
BLEEDING	5	0.8	0.8	1	0.6	0.9	1	
PATCHING	5	0.3	0.6	1	0.6	0.8	1	
DEBONDING	5	0.4	0.7	1	0.5	0.8	1	
CRACK SEALING DEFICIENCY	5	1	1	1	0.5	0.8	1	
RUTTING	10	0.3	0.7	1	0.6	0.8	1 **T**	
SETTLEMENT	0	0.0	0.0	0.0	0.0	0.0	0.0	
POTHOLES	10	0.4	0.8	1	0.5	0.8	1 **T**	
WHEEL TRACK CRACKING	15	0.4	0.7	1	0.5	0.7	1 **T**	
BLOCK AND TRANSVERSE CRACKING	10	0.4	0.7	1	0.5	0.7	1	
LONGITUDINAL CRACKING	5	0.4	0.7	1	0.5	0.7	1 **T**	
EDGE CRACKING	10	0.4	0.7	1	0.5	0.7	1 **T**	
THERMAL CRACKING	10	0.4	0.7	1	0.5	0.7	1	

*L = LOW　**O = OCCASIONAL　TOTAL DEDUCT =
M = MEDIUM　F = FREQUENT　SUM OF STRUCTURAL DEDUCT (**T**) =
H = HIGH　E = EXTENSIVE　**100 - TOTAL DEDUCT = PCR =**

Figure 11-4 ODOT flexible Pavement Condition Rating Form.

A spreadsheet containing ratings for state highway segments collected from 1985 to 2015 was provided by ODOT. The data for the study sections were extracted and used to generate Table 11–8. Data extraction for a section was discontinued when the PCR records indicated the section was rehabilitated. Sargand et. al [2010] found the following linear equation predicted the average performance of flexible pavement in Ohio:

$$PCR=98.6-1.55*Age$$

Where PCR=Pavement Condition Rating and Age = age of pavement in years

Sargand et al [2010] also found flexible pavements with exceptional performance had an average PCR predicted by the following linear equation:

$$PCR=103-1.32*Age$$

These equations were used to divide the sections in this study into three categories: exceptional performance, shown in Figure 11-5; average performance, shown in Figure 11-6; and poor performance, shown in Figure 11-7.

Table 11–9. Pavement Condition Rating (PCR) history.

County-Route-Section	Construction Completion Year	Age (yr)																	
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
		PCR	PCR	PCR	PCR	PCR	PCR	PCR	PCR	PCR	PCR	PCR	PCR	PCR	PCR	PCR	PCR	PCR	PCR
COS-36-20.83	1998	99	99	95	94	94	93												
CUY-71-0.00	1999	100	90	99	98	94	94	94	94	92	89	90	89	87	86	86	85	83	
FUL-20-6.75	2000	97	98	96	93	92	92	87	89	89	88	84	82	80		72			
HAM-264-6.87	2000	74	74	74	74	70	68	68											
WOO-795-2.01	2000	100	95	89	87														
FAY-35-2.57	2001	98	94	94	92	89	89	84	86	86	85	85	83						
GRE/FAY-35-26.20/0.00	2001	99	98	97	97	95	95	94	94	94	94	94	94	93					
LUC-2-21.24	2001	100	96	95	94	94	94	94	91	90	88	93	93	92	92	89	87		
VIN-50-11.75	2001	99	99	96	94	91	90	87	81	79									
WAR-71-3.78	2001	100	99	95	95	95	93	93	90	88	88	84	81	80	80	78	77		
FAI-33-7.31	2003	99	98	97	98	97	95	92	87	82	79	77	74	69					
FAI-33-13.25	2003				99	97	97	97	95	92	89	87	83	76					
MAD-142-0.49	2003	99	98	96	96	96	95	95	92	88	82	82	81	81					
FAI-33-17.44	2004			99	95	95	95	93	89	89	87	85	81						
ASD-39-0.00	2005		99	96	88	88	82	71	71	66	63	57							
FAI-33-19.79	2005		99	98	97	97	92	94	92	91	88	86							
MER-219-8.72	2005	100	99	98	97	89	86	84	84	83	81	81							
ROS-207-0.00	2006	99	99	98	97	97	96	94	94	91	91								
WAY-30-11.86	2006		99	98	98	93	88	85	83	76	74								
WAY-30-11.86	2006		98	96	96	94	94	94	94	94	94								
HAN/WYA-30-3.00/0.00	2007		99	96	93	90	89	90	83	97									
MED-71-5.94	2007	99	99	98	96	96	95	92	87	84									
CLI-73-12.03	2008	99	99	97	96	96	96	91	89										
DEF-24-4.94	2008	100	99	96	95	91	91	88	86										
FRA/LIC-161-23.30/0.00	2008		99	97	96	95	94	91	89										
SUM-77-13.54	2008	100	97	99	98	98	97	95	95	90	90								
BUT-75-5.91	2010	98	98	98	96	95	94												
CLI-73-6.52	2011		99	98	91	95													
CLA-70-13.98	2012	100	99	98	97	97													
LAK-2-7.60	2012	100	96	94	92														
MRW-71-3.17	2015	100	100	99															

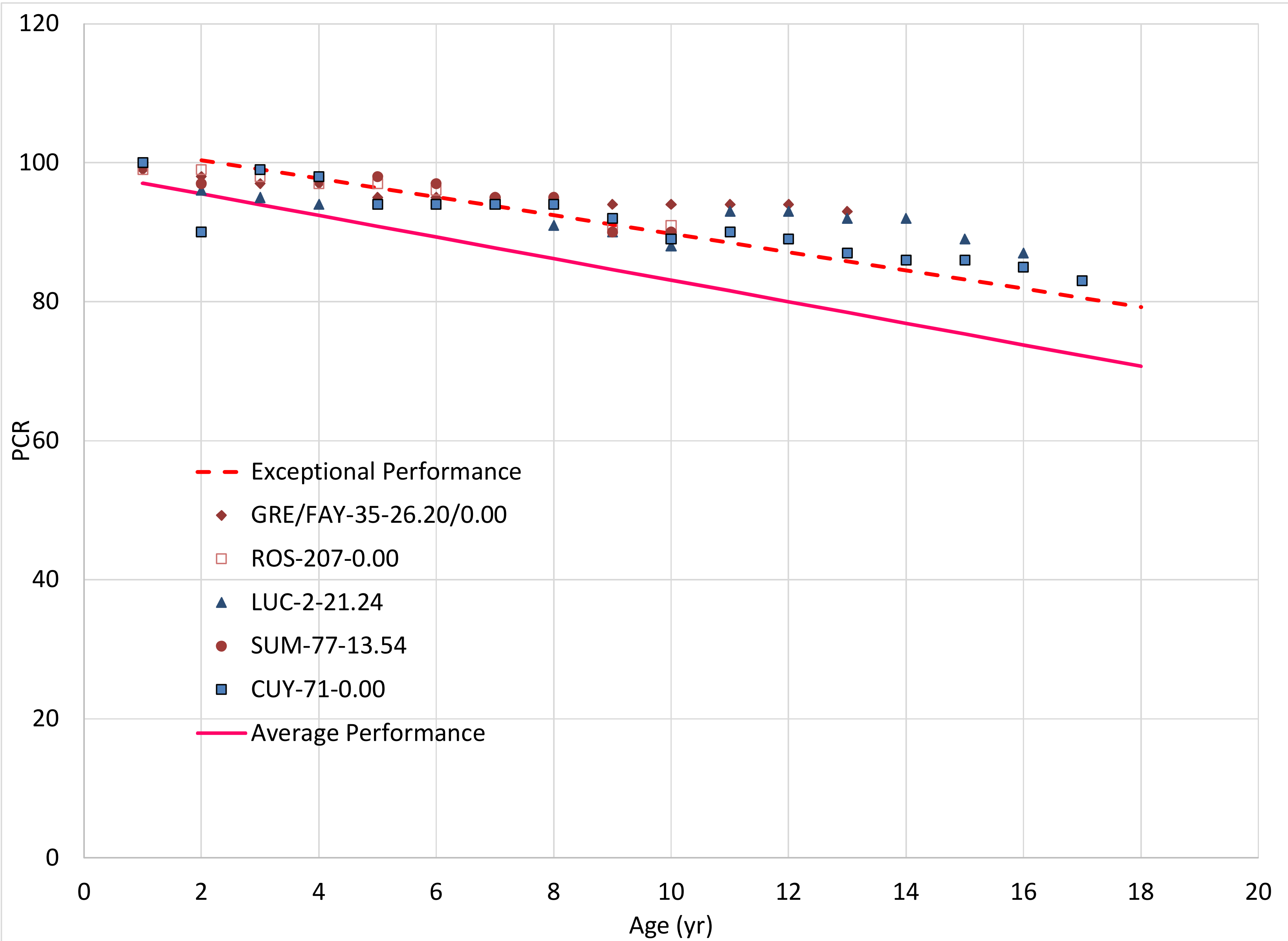


Figure 11-5. Projects with exceptional PCR performance.

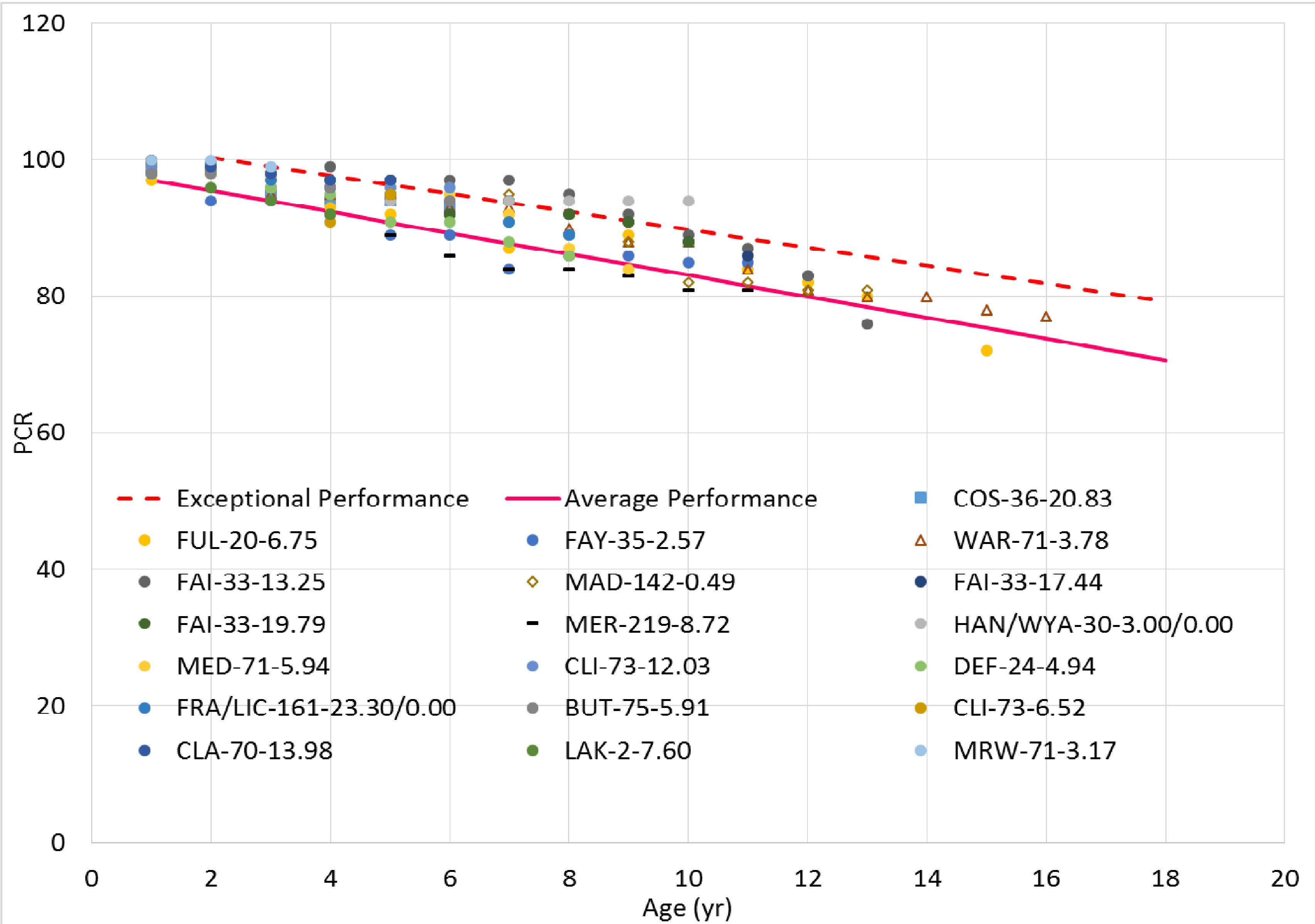


Figure 11-6. Projects with average PCR performance.

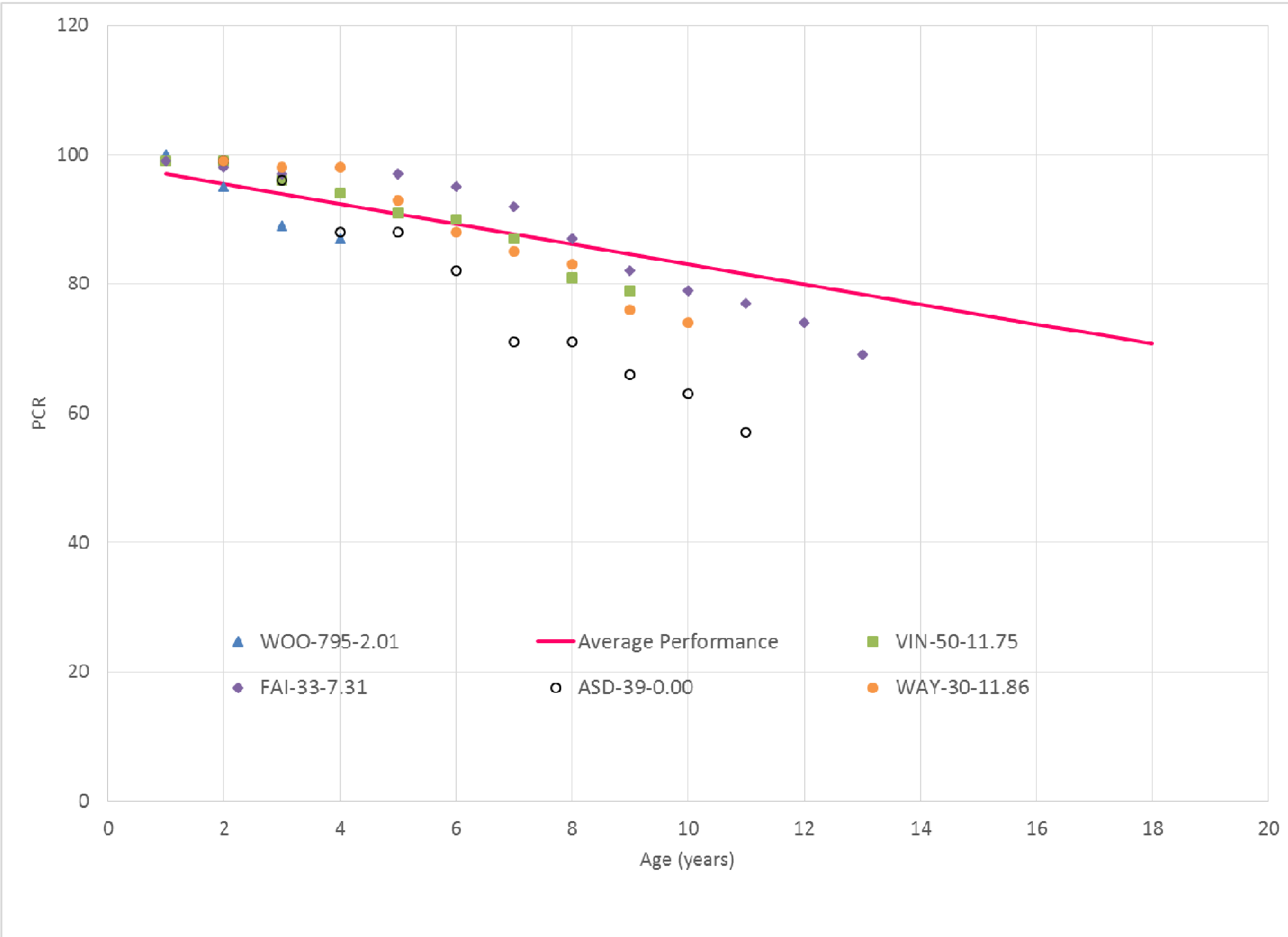


Figure 11-7. Projects with poor PCR performance.

Three of the sections with exceptional performance, LUC-2, ROS-207, and SUM-77, had design truck traffic much lower than predicted truck traffic, which may contribute to their exceptional performance. Three of the poor performers, ASD-39, VIN-50, and FAY-35, experienced truck traffic greater than predicted by a factor of 1.3 to 9.3, which may explain the poor performance of these sections. As noted later, segregation was observed in some core samples extracted from FAY-35 which may have also contributed to the poor performance.

The distresses present in poor performance projects as well as the projects which were overlaid at least within 15 years of construction are shown in Table 11–10. A key to the PCR distress rating codes used in the table are in Table 11–12; distress columns with bold font are those considered structural by ODOT.

WAY-30 was constructed as a demonstration of the perpetual pavement concept. The surface layer was a stone mastic asphalt (SMA) which did not perform well as evidenced by the distresses; raveling, patching, surface disintegration, and block cracking.

The remaining projects were performing poorly or were overlaid for a combination of functional and structural distresses with ASD-39, FAI-33-7.31, and WOO-795 showing the highest structural deduct values.

Segregation occurring at the end of the asphalt load was observed on FRA-71-5.29 during sampling for this research shown in Figure 11-8 through Figure 11-10. Core samples obtained from additional project sites also appeared to show evidence of segregation. For seven project sites, segregation was observed in at least one-third of the collected cores. Examples of this are provided below for DEF-24, FAY-35, and GRE-FAY-35 in Figure 11-11. Projects with segregation visible in the cores or on the surface of the base layer, and their current PCR, where available, are shown in Table 11–11. As before, a key to the PCR distress rating codes used in the table are in Table 11–12; distress columns with bold font are those considered structural by ODOT. With the exception of FAY-35-2.57, which also appears in Table 11–10, due to poor performance leading to an overlay, the sections with segregation visible in the cores have not shown much surface distress. Due to the nature of segregation and the random sampling of the cores, identifying projects as segregated or not segregated was not feasible for this study. However, based on the number of cores in which segregation was observed, it is recommended that control of segregation be further investigated.

Figure 11-8. End of load segregation.

Figure 11-9. Area of excessive fine aggregate due to end of load segregation.

Figure 11-10. Area of excessive coarse aggregate due to end of load segregation.

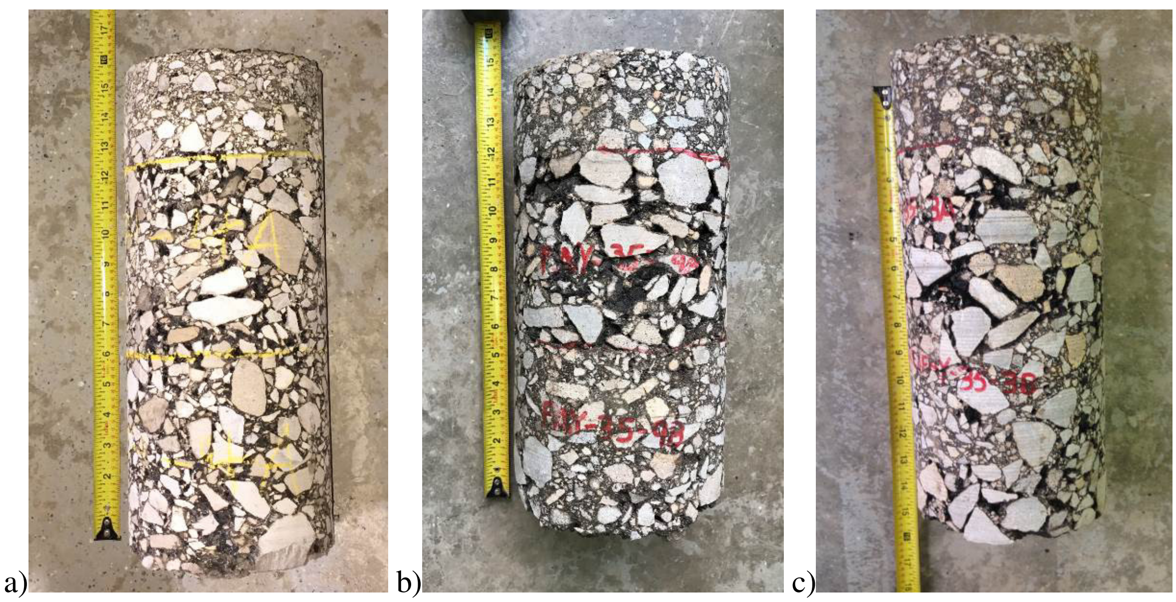


Figure 11-11. Examples of segregation in cores from: a) DEF-24, b) FAY-35, and c) GRE-FAY-35.

Table 11–10. Distresses present in projects with poor performance and/or overlaid within 15 years of construction. Items in bold represent structural distresses. For key to PCR distress codes, see Table 11–12.

County-Route-Section	ODOT CMS Item	Direction	Poor PCR Performance	Identified by District	Early Overlay	Completion date	PCR date	Age at time of PCR (yr)	PCR	Structural deduction	Raveling	Patching	Surface disintegration or debonding	Crack seal deficiency	**Rutting**	Settlement	**Wheel track cracking**	Block cracking	**Longitudinal joint cracking**	**Edge cracking**	Random cracking	Thermal cracking
ASD-39-0.00	302	EB	x			2005	2015	10	57	21	MF			F	**MO**	LO	**HO**	HE	**HO**	**HF**		MO
FAI-33-7.31	880	SB	x	x	x	2003	2015	12	69	11	LE	HO		F	**LE**		**MO**	HF	**MF**			MO
VIN-50-11.75	301	EB	x		x	2001	2010	9	79	8	LE	MO						LE	**LF**		MO	
WAY-30-11.86	302	WB	x		x	2006	2015	9	74	8	MF	MF	LF	E			**LF**	HO	**HF**			
WOO-795-2.01	302	EB	x		x	2000	2011	11	76	9	LE	LO		E	**LE**		**LF**	HO	**ME**			MF
FAI-33-13.25	880	SB		x	x	2003	2015	12	76	8	LE	MO		F	**LO**	LO	**LF**	HF	**MF**			
FAY-35-2.57	880	EB		x	x	2001	2015	14	83	5	MO			F	**LO**	MO		HO	**ME**			
FAI-33-17.44	880	SB			x	2004	2015	11	81	7	LE			F		LO	**LF**	MO	**MF**			LO
FAI-33-19.79	880	SB			x	2005	2015	10	86	6	LF			O			**LO**	MO	**MF**			

Table 11–11. Projects with segregation visible in the cores or on the surface of the base layer during construction. Items in bold represent structural distresses. For key to PCR distress codes, see Table 11–12.

County-Route-Section	ODOT CMS Item	Direction	Cores visibly segregated	Segregation visible on base surface	Completion date	PCR date	Age at time of PCR (yr)	PCR	Structural deduction	Raveling	Patching	Surface disintegration or debonding	Crack seal deficiency	**Rutting**	Settlement	**Wheel track cracking**	Block cracking	**Longitudinal joint cracking**	**Edge cracking**	Random cracking	Thermal cracking
FAY-35-2.57	880	EB	x		2001	2015	14	83	5	MO			F	**LO**	MO		HO	**ME**			
CLI-73-6.52	880	EB	x		2011	2015	4	95	1	LF							LO	**LO**			
DEF-24-7.96	880	EB	x		2008	2015	7	86	6	MO	LO		O	**MO**			LO	**MO**			
FRA/LIC-161-14.41	880	EB	x		2008	2015	7	89	6.8	LF				**LO**		**LO**	LO	**LE**			
GRE/FAY-35-26.20/0.00	301	EB	x		2001	2015	14	81	5.3	MO			F	**LO**			HF	**ME**			
MER-219-8.72	302	NB	x		2005	2015	10	81	8.4	LF			E	**LO**		**LF**	MO	**MF**			
WOO-75-19.43	301	SB	x		2017	Active construction project, no PCR data available															
FRA-71-5.29	302	NB		x	2018	Active construction project, no PCR data available															

Table 11–12. Key to PCR distress rating codes in previous two tables.

Distress Rating (PCR) key			
First letter = Severity		Second letter = Extent	
L	Low	O	Occasional
M	Medium	F	Frequent
H	High	E	Extensive

Appendix F: Project Information

Information specific to the project were summarized and used for further evaluating the laboratory test results and pavement performance. Pavement age was determined for each project based on the year in which cores were obtained and the approximate year of construction of the asphalt base layers. Plan information and approved JMFs were reviewed for each project. The CMS year, base type (ODOT Item 301, 302, 880, 301 as per plan (APP), or where both Items 301 and 302 were specified (301/302)), and base thickness specified in the plans were noted for each project for which plans were obtained. Since several projects had multiple JMFs approved, all JMFs (or JMF information obtained from ODOT's database) were reviewed and the following information was noted for each JMF, as shown in Table 11-13:

- Base type (ODOT Item 301 or Item 302),
- Coarse aggregate type and amount
- Amount of RAP,
- Amount of RAS, and
- Amount of virgin binder and total binder.
- PG of virgin binder
- Tensile strength ratio

Where multiple JMFs were approved for a project, determining which JMF was actually used, or constructed within the sampling limits was not possible for all projects, therefore, more general information was summarized based on the JMFs approved for the project. For instance, if all JMFs were for an ODOT Item 301 mix, then the mix type used was classified as "301". Where multiple mix types were approved for a project, the mix type was classified as unknown, "UNK", unless district personnel was able to identify the specific JMF used. A similar logic was used for classifying the aggregate type (a blend of limestone and gravel (blend), limestone (LMS), or gravel (GRVL)), and whether RAP or RAS was used in the project. This information, along with pavement age, district and region the project is in, and the plan information is summarized in Table 11-14. Due to the differences in RAP and/or RAS content among the various JMFs for each project the amount of recycled material could not be determined for each project, nor, could the binder content be determined. Although the JMF used for each project could not be identified, reviewing the binder contents for Item 301 and Item 302 JMFs revealed that in general, Item 301 base mixes which ranged from 4.6 to 5.0% tended to have greater binder contents than Item 302 mixes which ranged from 3.2 to 4.9%.

Table 11–13 Details for Approved JMFs

Proj ID	Project #	PID	Sale Date	JMF	Virgin Binder PG	JMF Type	TSR %	RAP %	RAS %	Virgin Binder %	Total Binder %	Coarse Aggregate: LMS %	Coarse Aggregate: GRVL %
ASD-39-0.00	2004-0500	23578	9/22/2004	B322193	64-22	302		25		2.7	4.0	35	30
				B322214	64-22	302	72.2	20		2.8	4.0	35	30
ATH-33-11.74	2017-0008	84468	1/19/2017	B170263	64-22	302		25		2.9	4.2	25	35
				B170879	64-22	302		25		2.8	4.2	25	35
BUT-75-5.91	2008-0246	75971	4/2/2008	W322432	64-22	302		40		2.5	4.1	33	15
				W322461	64-22	302		40		2.4	4.1	35	12
CLA-70-13.98	2010-0243	84664	5/27/2010	W322458	58-28	302	85.9	40		2.1	3.9	40	11
CLI-73-6.52	2009-0244	78571	5/6/2009	W313057	58-28	301		40		3.0	5.0	39	
				W312661	58-28	301		40		3.2	5.0	40	
CLI-73-12.03	2006-0413	78569	10/4/2006	B322337 (warranty only)	58-28	302		40		2.6	4.4	55	
				B322351	58-28	302		40		2.6	4.4	55	
COS-36-20.83	1996-0278	14142	4/24/1996	B321767	58-28	302		30		1.6	3.4	30	25
				B321756	64-22	302		0		3.4	3.4	20	50
CUY-71-0.00	1998-0748	15717		B321815	64-22	302		30		1.9	3.4	55	
CUY/SUM-77-0.00/32.73	2016-3019	79671	5/26/2016	B171008	58-28	302	75.4	40		2.1	4.0	50	
				B171147	58-28	302	75.8	40		2.2	4.1	40	10
				B170794	58-28	302	72.3	40		2.2	4.1	50	
				B170893	58-28	302	73.7	40		1.8	4.6	51	
DEF-24-7.96	2006-0087	24337	3/22/2006	B322376	58-28	302		0	10	2.4	4.1	82	
DEL-23-17.64	2012-0284	79370	5/24/2012	B120958	64-22	302	90.4	20		3.2	4.2	65	
DEL-23-19.24	1997-0335	16350	5/21/1997	B321759	64-22	302		0		3.5	3.5	75	
FAI-33-7.31	2001-0136	16293	3/20/2001	B321953	58-28	302		40		2.1	3.7	13	29
				B321958	58-28	302		40		1.6	3.8	14	28
FAI-33-13.25	2002-0110	16294	3/6/2002	B312267	64-22	301		25		3.3	4.7		50
				B312216	64-22	301		20		3.5	4.7		60

Proj ID	Project #	PID	Sale Date	JMF	Virgin Binder PG	JMF Type	TSR %	RAP %	RAS %	Virgin Binder %	Total Binder %	Coarse Aggregate: LMS %	Coarse Aggregate: GRVL %
FAI-33-13.25	2002-0110	16294	3/6/2002	B311112	64-22	301		0		4.7	4.7		65
				B322050	64-22	301		20		3.5	4.6	50	
FAI-33-17.44 SR 13	2002-0446	16295	10/23/2002	B312206	58-28	301		40		2.9	4.7		44
				B312267	64-22	301		25		3.3	4.7		50
FAI-33-17.44	2002-0446	16295	10/23/2002	B322013	58-28	302		40		1.8	3.6	14	28
				B322107	58-28	302		40		2.4	3.8	14	26
FAI-33-19.79 SR 18	2003-0046	23057	1/17/2003	B312267	64-22	301		25		3.3	4.7		50
FAI-33-19.79	2003-0046	23057	1/17/2003	B322200	58-28	302		35		1.9	3.8	13	26
				B322121	64-22	302		20		2.9	4.1	60	
				B322206	64-22	302		20		3.3	4.4	55	
FAY-35-2.57	2000-0577	9078	2/13/2000	B321923	64-22	302		20		2.4	3.4	65	
				B321921	64-22	302		20		3.4	4.3	62	
				B319506	64-22	301		20		4.1	5.0	52	
				B321925	64-22	302		20		2.2	3.2	65	
FRA-70-3.41	2015-0396	25594	8/6/2015	B160044	58-28	302		40		3.2	4.9	52	
				B160464	58-28	302	85.7	40		2.6	4.4	20	20
				B160551	58-28	302	88.6	40		2.3	4.4	22	18
				B160888	58-28	302	91.6	40		2.5	4.4	22	18
				B160749	58-28	302		40		2.9	4.9	52	
				B170456	58-28	302		40		2.6	4.9	52	
FRA-71-5.29	2015-0395	84868	7/23/2015	B170145	58-28	302		35		2.5	4.3	34	19
FRA/LIC-161-23.20/0.00	2006-0150	24486	4/28/2006	B312739	58-28	301		40		2.6	4.8	50	
				B312738	58-28	301		40		2.8	5.0	50	
FRA-270-21.67	2015-0249	81747	5/29/2015	B170145	58-28	302		35		2.5	4.3	34	19
FRA-270-35.41 Part 2	2017-0003	84620	2/16/2017	B170145	58-28	302		35		2.5	4.3	34	19
FUL-20-10.86	2000-0341	19342	6/7/2000	B321856	58-28	302		25		2.3	3.7	60	

Proj ID	Project #	PID	Sale Date	JMF	Virgin Binder PG	JMF Type	TSR %	RAP %	RAS %	Virgin Binder %	Total Binder %	Coarse Aggregate: LMS %	Coarse Aggregate: GRVL %
GAL-35-8.32	2007-0334	22520	6/27/2007	B311991	64-22	301		0		4.7	4.7		60
GRE/FAY-35-26.20/0.00	2000-0091	4388	3/8/2000	B312090	58-28	301		20	8	2.8	5.0	49	
				B310027	AC-20	301		0		4.7	4.7		58
				B321923	64-22	302		20		2.4	3.4	65	
				B319506	64-22	301		20		4.1	5.0	52	
				B317500	64-22	301		15		4.0	4.7		80
				B321921	64-22	302		20		3.4	4.3	62	
HAM-264-6.87	2000-0135	13853	3/15/2000	Unknown									
HAN/WOO-75-19.22/0.00	2014-3000	95437	2/18/2014	B160170	58-28	302		40	5	1.8	4.1	49	
				B160168	58-28	302		40	5	1.8	4.1	49	
HAN-30-3.00	2005-0003	77302	1/19/2005	B322274	58-28	302		5	6	2.6	3.9	66	
LAK-2-7.60	2010-0215	79545	5/6/2010	W101056	64-22	302		40		1.8	3.8	48	
				W100049	64-22	302		40		1.8	3.8	48	
LIC-161-1.83	2016-8030	97879	8/25/2016	B170259	58-28	301		40		2.9	5.0	42	
LUC-2-21.24	1999-0141	9159	3/17/1999	B321859	58-28	302		30		2.3	3.6	55	
				B321823	58-28	302		30		2.4	3.7	50	
				B321811	58-28	302		30		1.8	3.5	65	
LUC-75-6.70	2014-0536	76032	12/18/2014	B151035	58-28	302	76.1	40		2.0	4.0	49	
				B160994	58-28	302	73.6	40		2.0	4.1	48	
				B160777	58-28	302	72.9	40		2.3	4.0	48	
				W141129	58-28	302	76.1	40		2.0	4.0	48	
				B170528	58-28	302	72.8	40		1.8	4.0	50	
				W151035	58-28	302	76.1	40		2.0	4.0	49	
MAD-142-0.49	2001-0317	11739	6/20/2001	B312240	64-22	301		30		3.6	5.0	50	
				B311218	AC-20	301		0		5.0	5.0	66	
				B318546	64-22	301		30		3.7	5.0	15	35

Proj ID	Project #	PID	Sale Date	JMF	Virgin Binder PG	JMF Type	TSR %	RAP %	RAS %	Virgin Binder %	Total Binder %	Coarse Aggregate: LMS %	Coarse Aggregate: GRVL %
MAD-142-0.49	2001-0317	11739	6/20/2001	B312189	64-22	301		30		3.6	5.0	50	
MED-42-17.68	2016-0430	92954	7/28/2016	Unknown									
MED-71-9.56	2005-0343	14018	8/10/2005	B322233	58-28	302		40		1.7	3.8	38	
				B312576	58-28	301		45		2.7	5.0	38	
MER-219-14.04 "302"	2005-0313	19968	5/25/2005	B322238	64-22	302		0		3.9	3.9	65	
				B322236	64-22	302		0		3.9	3.9	67	
MER-219-14.04 "301"	2005-0313	19968	5/25/2005	B312557	64-22	301		0	5	4.1	5.0	60	
				B311390	64-22	301		0		5.0	5.0	59	
MRW-71-3.17	2013-3001	86920	2/14/2013	Unknown									
ROS-207-0.00	2004-0533	18492	11/3/2004	B312297	64-22	301		25		3.3	4.7		50
SUM-77-17.20	2006-0151	16514	4/12/2006	B322322	64-22	302	78.7	25		2.8	4.1	57	
VIN-50-11.75	2001-0123	10504	3/7/2001	B311024	64-22	301		0		4.7	4.7		55
WAR-71-3.78	1999-0780	10696	12/17/1999	Unknown									
WAR-71-14.20	2010-0280	22950	5/27/2010	B160787	58-28	302	90	45		2.1	4.0	42	3
				B141000	58-28	302	89.9	45		1.8	3.9	40	10
				W110949	58-28	302	87.8	45		1.9	3.9	35	10
WAY-30-11.86 "301"	2004-0044	16285/16287	2/20/2004	B312539	64-22	301		20		3.4	4.6	55	
WAY-30-11.86 "302"	2004-0044	16285/16287	2/20/2004	B322255	64-22	302	75	20		3.0	4.0	62	
				B322228	64-22	302		20		2.8	4.0	63	
WOO-75-10.61	2014-0170	95435	4/10/2014	B/W141129	58-28	302	76.1	40		2.0	4.0	48	
				B/W151035	58-28	302	76.1	40		2.0	4.0	49	
				B160777	58-28	302	72.9	40		2.3	4.0	48	
				B150632	58-28	302	71.7	40		2.5	4.1	50	
				W120217	58-28	302	73.1	40		2.1	3.9	50	
WOO-75-19.43	2014-0237	25521	5/15/2014	B150327	58-28	301		40		2.9	5.0	45	
				B160861	58-28	302		40		2.1	4.0	54	

Proj ID	Project #	PID	Sale Date	JMF	Virgin Binder PG	JMF Type	TSR %	RAP %	RAS %	Virgin Binder %	Total Binder %	Coarse Aggregate: LMS %	Coarse Aggregate: GRVL %
WOO-75-19.43	2014-0237	25521	5/15/2014	B150328	64-22	301		29		3.5	5.0	51	
				B170080	64-22	301		30		3.8	5.0	43	
				W140564	64-22	301		40		3.6	5.0	35	
				B170055	58-28	302		40		2.4	4.0	52	
				B170126	58-28	301		40		3.4	5.0	36	
WOO-75-2.37	2014-0199	95436	4/24/2014	B150567	58-28	302		45		2.0	4.1	49	
				B150407	58-28	302		40	5	1.8	4.2	46	
				B160169	58-28	302		40	5	1.9	4.2	45	
				B160167	58-28	302		40	5	1.8	4.1	45	
				B160203	58-28	302		40		1.8	4.0	48	
				B160366	58-28	302		40		2.2	4.2	50	
				B160170	58-28	302		40	5	1.8	4.1	49	
				B140943	58-28	302		39		1.8	4.2	41	
				B150568	58-28	302		45		2.0	4.1	49	
				B150409	58-28	302		40	5	1.8	4.2	47	
				B150073	58-28	302		38	6	1.8	4.2	46	
				B160373	58-28	302		40		2.0	4.1	53	
WOO-795-2.01	1999-0505	13725	6/9/1999	B321827	64-22	302		20		3.0	3.9	67	
				B321821	64-22	302		15		3.1	3.9	73	

Table 11–14. Summary of plan and JMF information for each project

Project ID	Region	District	Spec Year	Plan Base Type	Plan Base Thick (in)	Plan Base Thick (mm)	Classify Mix Type Used	Classify Aggregate Type Used	RAP Used	RAS Used	Pavement Age	Pavement Performance	Measured to Predicted ADTT
ASD-39-0.00	NE	3	2002	302, APP	8	203	302	Blend	Y	N	12	Poor	1.32
ATH-33-11.74	SE	10	2016	302	5.5	140	302	Blend	Y	N	0	N/A	N/A
BUT-75-5.91	SW	8	2005	302	9	229	302	Blend	Y	N	7	Average	0.77
CLA-70-13.98	SW	7	2008	302	10.5	267	302	Blend	Y	N	4	Average	0.89
CLI-73-6.52	SW	8	2008	880	11	279	301	LMS	Y	N	5	Average	0.30
CLI-73-12.03	SW	8	2005	880	10.5	267	302	LMS	Y	N	9	Average	0.25
COS-36-20.83	Central	5	1995	302	8	203	302	Blend	UNK	N	19	Average	0.56
CUY-71-0.00	NE	12	1997	302	11.25	286	302	LMS	Y	N	18	Exceptional	0.93
CUY/SUM-77-0.00/32.73	NE	12	2013	302	7.5	191	302	UNK	Y	N	0	N/A	N/A
DEF-24-7.96	NW	1	2005	880	11	279	302	LMS	N	Y	9	Average	UNK
DEL-23-17.64	Central	6	2010	302	4	102	302	LMS	Y	N	5	N/A	N/A
DEL-23-19.24	Central	6	1995	302	12	305	302	LMS	N	N	20	N/A	0.77
FAI-33-7.31	Central	5	1997	880	9	229	302	Blend	Y	N	14	Poor	0.90
FAI-33-13.25	Central	5	1997	880	8	203	301	GRVL	UNK	N	14	Poor	0.70
FAI-33-17.44 SR 13	Central	5	1997	301	6	152	301	GRVL	Y	N	13	N/A	UNK
FAI-33-17.44	Central	5	1997	880	7	178	302	Blend	Y	N	13	Poor	2.36
FAI-33-19.79 SR 18	Central	5	1997	301	6	152	301	GRVL	Y	N	12	N/A	UNK
FAI-33-19.79	Central	5	1997	880	7	178	302	LMS	Y	N	12	Poor	2.36
FAY-35-2.57	Central	6	1997	880	12	305	UNK	LMS	Y	N	16	Poor	9.35
FRA-70-3.41	Central	6	2013	302	12	305	302	UNK	Y	N	0	N/A	N/A
FRA-71-5.29	Central	6	2013	302	12	305	302	Blend	Y	N	0	N/A	N/A
FRA/LIC-161-23.20/0.00	Central	6	2005	880	7.5	191	301	LMS	Y	N	9	Average	0.82
FRA-270-21.67	Central	6	2013	302	10	254	302	Blend	Y	N	0	N/A	N/A
FRA-270-35.41 Part 2	Central	6	2016	302	10.5	267	302	Blend	Y	N	0	N/A	N/A
FUL-20-10.86	NW	2	1997	302	12	305	302	LMS	Y	N	17	Average	0.64

Project ID	Region	District	Spec Year	Plan Base Type	Plan Base Thick (in)	Plan Base Thick (mm)	Classify Mix Type Used	Classify Aggregate Type Used	RAP Used	RAS Used	Pavement Age	Pavement Performance	Measured to Predicted ADTT
GAL-35-8.32	SE	10	2005	301	8	203	301	GRVL	N	N	10	N/A	N/A
GRE/FAY-35-26.20/0.00	Central	6	1997	301	11	279	UNK	LMS	Y	UNK	16	Exceptional	2.49
HAM-264-6.87	SW	8	1997	301/302	3/8	75/200	UNK	UNK	UNK	UNK	17	N/A	0.32
HAN/WOO-75-19.22/0.00	NW	2		302	12.5	318	302	LMS	Y	Y	0	N/A	N/A
HAN-30-3.00	NW	1	2002	880	15	390	302	LMS	Y	Y	10	Average	1.10
LAK-2-7.60	NE	12	2008	302	10	254	302	LMS	Y	N	5	Average	0.50
LIC-161-1.83	Central	5	2016	301	7	178	301	LMS	Y	N	0	N/A	N/A
LUC-2-21.24	NW	2	1997	301	10	254	302	LMS	Y	N	16	Exceptional	0.37
LUC-75-6.70	NW	2	2013	302	11.5	292	302	LMS	Y	N	0	N/A	N/A
MAD-142-0.49	Central	6	1997	301	8	203	301	LMS	UNK	N	14	Average	0.54
MED-42-17.68	NE	3	2013	301	6	152	UNK	UNK	UNK	UNK	0	N/A	N/A
MED-71-9.56	NE	3	2002	302	10.5	267	UNK	UNK	Y	N	10	Average	0.70
MER-219-14.04 "302"	NW	7	2002	302	7.5	191	302	LMS	N	N	12	Average	0.61
MER-219-14.04 "301"	NW	7	2002	301	3	76	301	LMS	N	UNK	12	Average	0.61
MRW-71-3.17	Central	6	2010	302	11	279	UNK	UNK	UNK	UNK	1	Average	0.55
ROS-207-0.00	SW	9	2002	880	8.5	216	301	GRVL	Y	N	11	Exceptional	0.35
SUM-77-17.20	NE	4	2005	880	9.5	234	302	LMS	Y	N	9	Exceptional	0.50
VIN-50-11.75	SE	10	1997	301	8	203	301	GRVL	N	N	16	Poor	1.77
WAR-71-3.78	SW	8	1997	302	12	300	UNK	UNK	UNK	UNK	16	Average	0.40
WAR-71-14.20	SW	8	2008	880	11.25	286	302	BLEND	Y	N	2	N/A	UNK
WAY-30-11.86 "301"	NE	3	2002	301, APP	3	76	301	LMS	Y	N	11	N/A	0.87
WAY-30-11.86 "302"	NE	3	2002	302	9/4 FRL	229/25	302	LMS	Y	N	11	Poor	0.87
WOO-75-10.61	NW	2	2013	302	11	279	302	LMS	Y	N	0	N/A	N/A
WOO-75-19.43	NW	2	2013	301	13	330	UNK	LMS	Y	N	0	N/A	N/A
WOO-75-2.37	NW	2	2013	302	11	279	302	LMS	Y	UNK	0	N/A	N/A
WOO-795-2.01	NW	2	1997	302	10	250	302	LMS	Y	N	17	Poor	0.14

Appendix G: Pavement thickness

The design procedure for asphalt pavement results in a design thickness for each layer. For construction contracts, asphalt plan quantity is given in cubic yards. However, compacted thickness is not easily measured or controlled during construction, so tonnage is monitored and used to estimate cubic yards, and therefore thickness. The sampling and testing plan was focused on material testing to estimate performance and did not include the measurement of thickness. However, the cores obtained from MAD-142 showed a high variability in thickness (see Figure 11-12). As a result, an evaluation of thickness variability was undertaken. Core thickness measurements taken in the field or estimated from core photographs are shown in Table 11–15 for the asphalt base and in Table 11–16 for total pavement thickness.

Figure 11-12. Pavement specimen cores from MAD-142.

As Bonaquist [2014] noted in his review of factors affecting pavement durability, the pavement must have sufficient thickness to carry the expected traffic. Base layer thickness and total thickness were estimated from core samples and compared with the base layer and total thickness specified in the plans. The cumulative frequency of the difference between planned base layer thickness and planned total thickness were plotted and shown in Figure 11-13. As shown in the plot, on average, total in-place pavement thickness was in agreement with total planned thickness while, slightly more than 50% of the core samples taken had in-place base thickness in agreement with the planned thickness. The plot also shows approximately 20% of the core samples may be at least 1 in (25 mm) deficient relative to the planned base thickness and the planned total thickness. While approximately the same percentage of core samples had 1 in (25 mm) additional thickness for each the base layer and total thickness, in-place thickness less than the designed thickness stands to have a greater impact. It should be noted samples were randomly selected within the sample limits and therefore variability associated with thickness should be primarily due to construction variability.

Table 11–15. Measured/estimated asphalt base thickness (1 in = 25.4 mm).

	Measured/Estimated Base Thickness (inch)																				Plan Thickness (inch)	Statistics			
Core Number / County-Route-Section	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20		Number of cores	average measured thickness (inch)	standard deviation	Coefficient of Variation
ATH-33-11.74	5.3	5.8		5.4	5.6		5.4	5.9	6.0	5.9		5.9	5.0	6.2	5.4	6.2					5.5	13	5.70	0.37	6.4%
MED-42-17.68	5.6	6.7	5.9			6.4		6.3	5.8	5.9	6.0	6.5	6.3	6.3							6.0	11	6.17	0.34	5.5%
FAI-33-17.44, SR-13	5.5		5.5	5.7	4.4	6.4	7.2		7.1	7.4	7.5		5.8	7.9							6.0	11	6.41	1.10	17.2%
FAI-33-19.79, SR-18	6.3	6.7	6.8	6.3	6.6	6.3	7.2	6.1	7.6	7.1	6.7	6.7	6.3	6.2							6.0	14	6.64	0.42	6.3%
LIC-161-1.83			6.8	6.7	7.8		7.6	6.4			7.6	7.3	6.1	7.2							7.0	9	7.06	0.59	8.4%
FAI-33-19.79	4.8	4.6	4.7	4.7	4.7	4.3	4.5	4.3	4.5	3.9		5.5	4.9	5.4							7.0	13	4.69	0.42	8.9%
FAI-33-17.44	6.3	6.2	5.6	5.6	6.3	5.8	5.9	5.8	5.8	5.3	5.2	7.4	5.5	6.1							7.0	14	5.92	0.54	9.2%
CUY/SUM-77-0.00/32.73	7.1				6.9	7.2	6.7	7.3				6.9		8.5							7.5	7	7.24	0.59	8.1%
FRA/LIC-161-23.20/0.00	8.8	8.6	9.0	8.8	8.4	7.9	7.5	7.3	7.2	7.5	8.0	7.7	7.4	7.5							7.5	14	7.97	0.62	7.8%
FAI-33-13.25		4.3	6.5	6.5	6.6	4.7	6.1	6.5	6.9	5.9	6.3	5.9	6.7	4.9							8.0	13	5.99	0.82	13.7%
ASD-39-0.00	8.2	7.7	7.0	8.3	8.7	6.0	6.7	8.5	7.5	7.6	7.8	9.4	9.1	9.3	8.5	8.5	7.3				8.0	17	7.99	0.93	11.7%
DEL-23-17.64		8.6	9.1	8.0	9.0	8.1	8.4	8.0	7.1	7.6	8.7	9.2	8.5	9.0							8.0	13	8.41	0.63	7.4%
MAD-142-0.49	8.7	8.5	12.8	9.9	8.0	8.7	8.1	8.3	7.7	5.9	8.2	8.6	8.5	8.7							8.0	14	8.61	1.48	17.2%
GAL-35-8.32	9.2	8.5	8.8		8.9	8.8	8.1	8.4	9.1	10.2		8.0	8.5	8.6							8.0	12	8.76	0.57	6.5%
VIN-50-11.75				8.7	9.2		8.8		9.1		9.0	8.9	9.0	8.9							8.0	8	8.95	0.18	2.0%
COS-36-20.83	8.7	9.4	9.3	9.6	9.1	8.9		9.3	8.7	9.6	8.9	8.9	7.3	9.4							8.0	13	8.99	0.62	6.9%
ROS-207-0.00	8.0	8.1	6.9	6.5	8.6	8.4	8.4	8.9	8.2	8.5	8.6	8.1	7.4	7.6							8.5	14	8.00	0.68	8.5%
FAI-33-7.31	8.9	8.8	10.0	8.6	8.8	9.3	8.3		8.1	8.7	7.4	7.6	6.8	7.5							9.0	13	8.38	0.87	10.4%
SUM-77-17.20			9.9	9.6						10.1	9.2										9.5	4	9.72	0.37	3.8%
BUT-75-5.91	8.6	8.6	8.7	8.8	8.3	8.9	9.5	8.9	9.1	8.6		8.2	8.9	8.5							9.9	13	8.74	0.34	3.9%
CUY-71-0.00		7.4		5.5	5.8	6.7	6.4	6.1	6.0	5.9			7.0								10.0	9	6.31	0.62	9.8%
LUC-2-21.24	11.3	9.5				10.8	12.1	10.4	9.8	10.7	11.6	9.3	9.9	9.9							10.0	11	10.48	0.91	8.7%
FRA-270-21.67	9.8	9.8	10.5	11.2	9.9	10.8	10.9	10.8	11.3	11.0	11.2	11.5	10.0	10.3							10.0	14	10.65	0.58	5.5%
LAK-2-7.60	11.3	11.5	10.8	11.5	11.3	11.0	10.0	9.8	9.8	9.5	10.5	12.0	11.5	11.0	9.5	10.0	10.0	10.8	10.3	11.5	10.0	20	10.66	0.77	7.3%
CLI-73-12.03		7.3			7.3	7.2		7.4													10.5	4	7.32	0.07	1.0%
MED-71-9.56									9.3												10.5	1	9.27		
MER-219-14.04	10.6		9.2		9.1		9.2				10.3			10.4							10.5	6	9.82	0.72	7.4%
FRA-270-35.41	11.6	11.5	11.2		11.3		11.5	11.3	11.4		10.8	11.0	11.0	10.7							10.5	11	11.20	0.30	2.7%
WOO-75-2.37			12.3	12.2	11.2			11.7	12.0	11.5	11.7		11.8	11.9	11.6						11.0	10	11.80	0.33	2.8%
DEF-24-7.96	11.5	11.9	11.3	11.5		11.2	10.6	11.0	11.8	10.7	10.3	10.6	10.6	10.8	10.7	10.5					11.0	15	11.01	0.50	4.5%
GRE/FAY-35-26.20/0.00	10.1	10.9		10.9	11.0			11.1			11.9	11.9	12.0	11.6							11.0	9	11.27	0.62	5.5%
WOO-75-10.61	11.5	11.7	11.2						10.7		11.7										11.0	5	11.35	0.39	3.5%
WAR-71-14.20		11.8	12.1		12.2	11.7			12.2	12.4	11.5	11.2	10.5	9.9							11.3	10	11.54	0.82	7.1%
LUC-75-6	10.5	10.4	10.4	11.0	11.4	12.5	12.5	12.1	12.4	11.0	12.2	11.8	11.7	12.1							11.5	14	11.58	0.77	6.7%
WAR-71-3.18		10.5		10.3	11.0		10.6	10.9					10.8	11.0							11.8	7	10.72	0.27	2.5%
FAY-35-2.57	9.2			10.3	11.1	10.8	9.5		9.3	10.2	10.7	10.4		10.1	10.2						12.0	11	10.15	0.62	6.1%
FUL-20-10.86		9.5														9.8					12.0	2	9.68	0.24	2.5%
FRA-70-3.41		11.7			11.8	11.2				12.0	12.3	11.6		11.2							12.0	7	11.68	0.40	3.4%
DEL-23-19.24	10.2	12.4	11.2	12.0	10.8	11.7		11.4	13.0	11.9	11.9	12.9	11.7								12.0	12	11.75	0.81	6.9%
FRA-71-5.29	12.8	12.8	10.8	12.5	11.8	12.3	12.0	12.0	11.8	11.8	12.0	12.5	12.0	12.0							12.0	14	12.05	0.51	4.2%
HAN/WOO-75-19.22/0.00	14.3	14.4	13.8	14.2	13.0	12.1		14.7	14.8	13.4	13.0	12.9		14.5							12.5	12	13.76	0.87	6.3%
WOO-75-19.43	12.6	12.2	12.5		14.0	13.7	14.1	14.3	12.6	12.5	13.5	13.3	13.6	13.3	14.2	13.5					13.0	15	13.33	0.69	5.2%
HAN-30-3.00	14.4																				15.4	1	14.41		

Table 11–16. Measured/estimated total asphalt pavement thickness (1 in = 25.4 mm).

	Measured/Estimated Total Pavement Thickness (inch)																				Plan Thickness (inch)	Statistics			
Core Number / County-Route-Section	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20		Number of cores	average measured thickness (inch)	standard deviation	Coefficient of Variation
MED-42-17.68	7.8	8.9	8.5			8.5		8.3	7.9	7.7	8.1	8.5	8.4	8.6							9.0	11	8.3	0.37	4.4%
FAI-33-19.79, SR-18	9.2	9.5	9.8	9.2	9.4	9.2	10.4	9.2	10.2	9.7	9.1	9.3	9.0	9.2							9.0	14	9.5	0.44	4.6%
FAI-33-17.44, SR-13	8.6		8.7	8.9	7.7	9.4	10.8		9.8	10.1	10.3		9.5	10.7							9.0	11	9.5	0.97	10.2%
GAL-35-8.32	10.5	9.8	10.3		10.3	10.0	9.4	9.6	10.2	11.6		9.6	9.4	9.8							9.5	12	10.0	0.62	6.2%
FAI-33-17.44	10.4	10.2	10.1	9.6	10.7	10.2	10.4	10.0	9.7	10.0	9.7	11.5	9.6	10.4							10.0	14	10.2	0.51	5.1%
FAI-33-19.79	10.3	10.7	10.8	10.8	10.4	10.4	10.8	9.6	10.4	11.0		11.5	10.4	10.9							10.0	13	10.6	0.44	4.1%
ASD-39-0.00	10.2	9.6	8.6	9.8	10.6	7.9	8.2	10.2	9.4	9.2	9.8	11.3	11.1	10.8	9.9	9.7	9.1				10.0	17	9.7	0.95	9.8%
CLI-73-12.03		10.6			10.8	10.8		10.7													10.5	4	10.7	0.11	1.0%
FRA/LIC-161-23.20/0.00	11.4	11.3	11.6	11.7	11.6	11.1	10.3	10.2	10.3	10.6	11.3	10.9	10.6	10.6							10.5	14	11.0	0.53	4.9%
CUY/SUM-77-0.00/32.73	9.5				9.0	9.3	8.8	9.0				8.9		10.3							10.8	7	9.3	0.53	5.7%
COS-36-20.83	10.2	11.2	11.1	11.4	10.8	10.4		10.8	10.4	11.4	10.6	10.4	8.9	11.0							11.0	13	10.7	0.67	6.3%
DEL-23-17.64		11.4	11.4	10.5	11.2	11.0	10.8	10.5	9.8	9.8	11.5	12.0	11.3	12.0							11.0	13	11.0	0.70	6.4%
MAD-142-0.49	11.9	12.8	15.7	13.1	10.9	12.0	12.0	11.7	11.3	9.7	12.3	12.5	11.9	12.9							11.0	14	12.2	1.34	11.0%
VIN-50-11.75				12.3	13.4		13.3		13.8		13.5	13.5	13.7	13.4							11.0	8	13.3	0.46	3.4%
FAI-33-13.25		9.3	11.6	11.6	11.5	10.8	11.8	12.6	13.0	11.6	11.4	11.6	12.0	9.8							11.3	13	11.4	1.00	8.8%
ROS-207-0.00	11.1	11.5	10.1	9.7	11.7	11.2	11.5	11.8	11.2	11.0	11.2	11.1	10.3	10.4							11.8	14	11.0	0.63	5.7%
FAY-35-2.57	12.0			13.4	13.6	13.6	12.7		11.5	12.9	13.3	13.0		13.0	13.4						12.0	11	12.9	0.67	5.2%
FAI-33-7.31	13.7	13.6	14.5	12.7	13.5	13.5	12.0		12.6	13.8	12.6	12.8	12.4	13.5							12.3	13	13.2	0.69	5.3%
SUM-77-17.20			13.0	12.9						13.0	12.4										12.8	4	12.8	0.29	2.3%
LUC-2-21.24	12.8	12.3				13.6	13.9	13.3	12.5	13.6	14.1	12.6	12.8	12.4							13.0	11	13.1	0.63	4.8%
BUT-75-5.91	11.0	11.0	11.9	11.3	11.1	11.2	12.5	11.7	11.9	11.4		12.1	11.5	11.4							13.2	13	11.5	0.46	4.0%
FRA-270-21.67	12.3	12.1	12.4	13.0																	13.3	4	12.4	0.41	3.3%
LAK-2-7.60	13.8	14.3	13.3	15.0	13.8	14.0	13.0	13.3	12.8	12.5	13.5	15.0	14.0	14.0	12.0	11.8	11.8	12.5	11.8	13.0	13.3	20	13.2	1.00	7.6%
CUY-71-0.00		10.9		9.0	9.5	10.6	10.2	9.5	9.0	9.4			10.0								13.3	9	9.8	0.69	7.0%
MER-219-14.04	14.1		13.8		13.1		12.3				15.4			14.1							13.5	6	13.8	1.05	7.6%
MED-71-9.56									14.3												13.8	1	14.3		
DEF-24-7.96	14.3	14.3	14.1	14.4		14.1	13.9	14.0	15.0	13.9	13.5	14.0	13.7	14.2	13.6	14.2					14.0	15	14.1	0.35	2.5%
GRE/FAY-35-26.20/0.00	13.6	14.1		13.6	14.6			14.2			14.6	14.9	14.9	14.4							14.0	9	14.3	0.48	3.3%
WOO-75-10.61	15.0	14.8	14.4						13.7		14.7										14.3	5	14.5	0.49	3.4%
WOO-75-2.37			15.2	15.4	15.2			15.1	15.5	16.0	15.0		14.8	15.5	16.8						14.3	10	15.4	0.58	3.7%
WAR-71-14.20		14.5	15.0		15.1	15.5			15.2	16.2	15.4	14.7	14.4	13.8							14.5	10	15.0	0.68	4.5%
LUC-75-6.70	12.3	12.4	11.3	12.3	13.5	14.0	14.2	14.0	13.9	12.9	13.9	14.6	14.1	13.9							14.8	14	13.4	0.96	7.2%
DEL-23-19.24	13.2	15.5	13.6	15.1	13.1	14.8		14.2	15.7	15.2	14.6	15.3	14.8								15.0	12	14.6	0.88	6.1%
FUL-20-10.86		15.0														14.0					15.3	2	14.5	0.68	4.7%
FRA-70-3.41		15.4			15.5	15.5				15.5	16.0	15.5		14.7							15.3	7	15.4	0.37	2.4%
HAN-30-3.00	17.1																				15.4	1	17.1		
HAN/WOO-75-19.22/0.00	17.7	18.0	16.6	17.1	16.0	15.6		17.3	17.7	17.1	16.1	16.6		17.5							15.8	12	16.9	0.75	4.4%
WOO-75-19.43	16.3	16.2	16.1		16.9	17.0	17.4	17.3	16.2	16.1	16.9	16.9	17.1	16.7	17.7	17.2					16.3	15	16.8	0.50	3.0%
WAR-71-3.18		15.6		15.5	16.1		15.2	15.6					15.6	15.9							16.3	7	15.6	0.26	1.7%

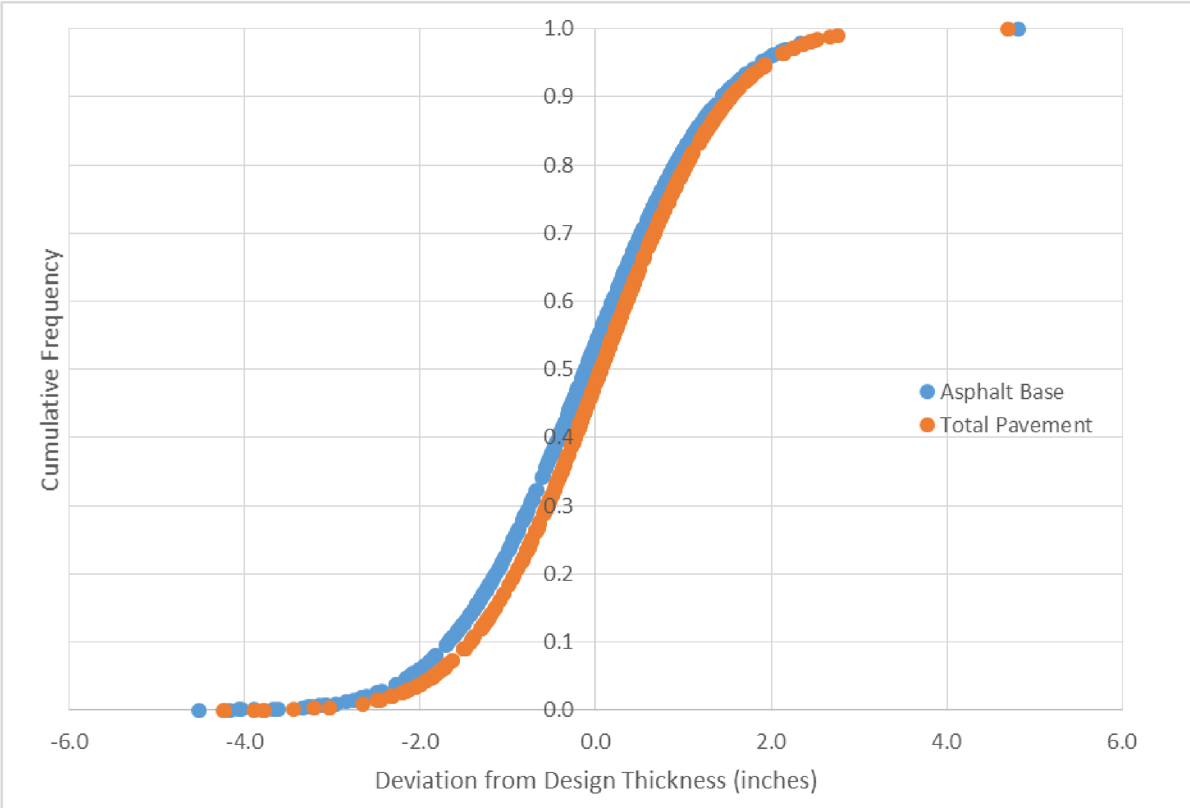


Figure 11-13. Cumulative frequency plot of deviation from plan thickness (1 in = 25.4 mm)

The in-place thickness relative to the design thickness was evaluated for each project. Where applicable, the base layer thickness was taken as the combination of the thickness of the top and bottom lifts of AC base. Shown below in Figure 11-14 and Figure 11-15 are the average measured or estimated in-place thickness for each project site for the base layer and the total pavement, respectively. The minimum and maximum thickness is reflected in the plot by the error bars. The design thickness is also plotted for comparison and they are plotted in increasing order of planned thickness. It should be noted that several projects were under construction at the time of sampling therefore the total in-place thickness is not known for those projects. There are several projects which had measured base thickness significantly less than the designed thickness. Additionally, there are several projects that exhibited a large spread in the base thickness. For some of those projects which had measured base thickness significantly less than the planned base thickness, the total measured thickness was close to planned total thickness. However, for others, the thickness difference was not made up in intermediate and surface layers. The final grade on some projects, such as MAD-142, is fixed by other features such as curb and gutter. The variation in thicknesses in asphalt pavement in these cases is likely due to errors in achieving the correct grade with the subgrade or aggregate base.

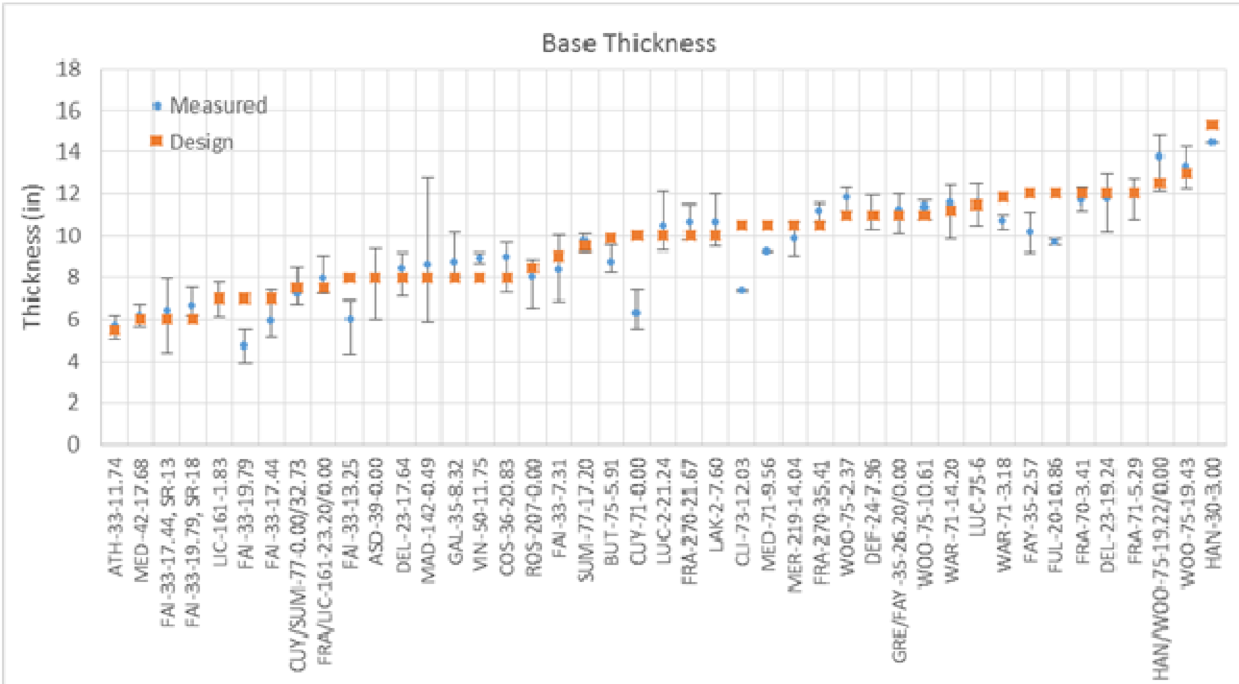


Figure 11-14. Measured/estimated base layer thickness (1 in = 25.4 mm).

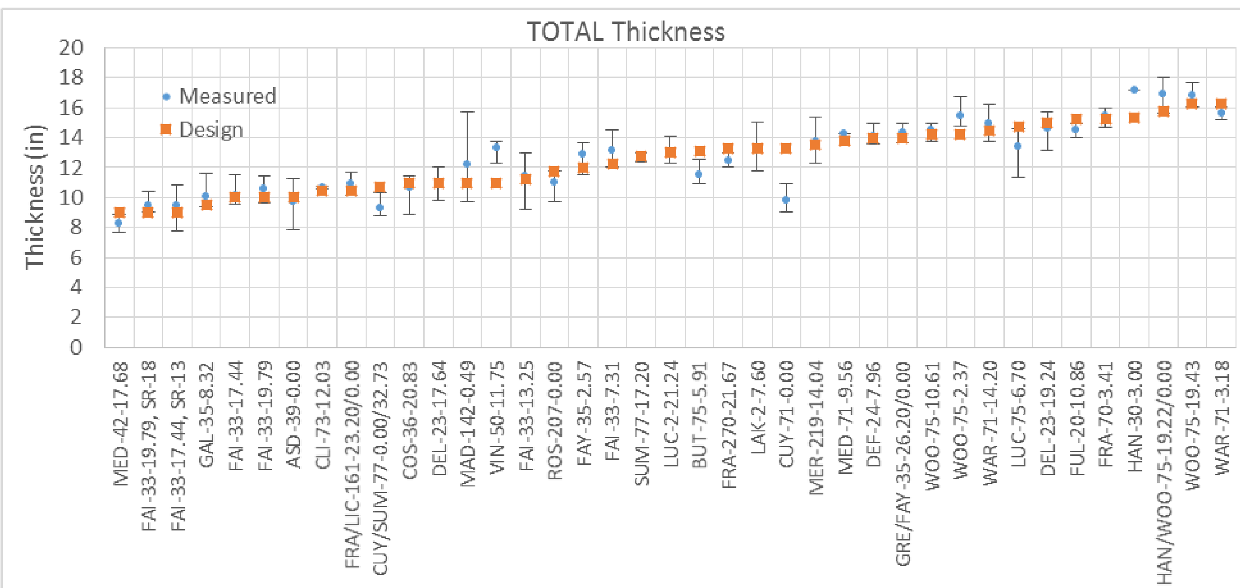


Figure 11-15. Measured/estimated total pavement thickness (1 in = 25.4 mm).

Appendix H: Laboratory Testing

Collection and preparation of field specimens for laboratory testing

The research team extracted 720 cores from 51 project sites located in 31 counties and 11 of the 12 ODOT Districts. For a majority of the projects, fourteen 6 in (152 mm) diameter cores were collected at each project site. The fourteen samples were cored in random locations distributed in a 1400 ft (427 ft) delimited segment at each project. Each core was labeled and the different layers were marked.

The core samples collected were brought to the Ohio University laboratory where pictures of each core were taken and the thickness of the core was registered. Each core was then separated using a saw into the different layers, typically: AC surface course, AC intermediate course, top lift of the AC base (labeled part A), and bottom lift of the AC base (labeled part B).

To be consistent among all project sites, the top lift of the AC base was selected for testing. In one case, the top lift was too thin to be used, therefore, testing for that project was conducted on the bottom AC base lift. After the bulk specific gravity (Gmb) of each base specimen was determined, it was dried, then the fourteen AC base specimens collected from each project site were separated into subsets of samples to be used for the different tests:

- Two were used to obtain the maximum theoretical specific gravity (Gmm) of the mix following AASHTO T 209;
- Three were tested for low-temperature cracking using the Semi-Circular Bending (SCB) test following the Illinois method described in AASHTO TP 124-16;
- Six were tested for moisture susceptibility to find the tensile strength ratio (TSR), following AASHTO T 283;
- Two were tested for Cantabro loss, following TEX-245-F;
- One specimen was kept as a back-up sample should any sample get damaged during the test specimen preparation process.

The selection of each subset was conducted randomly but the research team made an effort to obtain cores representative of the whole project site by avoiding areas where the asphalt mix may be influenced by other factors such as block outs, intersections, etc.

Laboratory Testing

Bulk specific gravity (Gmb) was determined for all AC base samples to determine in-place air voids of the top base lift. Gmb was determined following AASHTO T 166 prior to samples being cut and prepared for laboratory performance tests. Samples were then cut to the appropriate dimensions for Cantabro, SCB, and TSR tests and the Gmb was determined for each specimen and dried before testing commenced. For SCB samples, Gmb was determined prior to cutting the notch.

Maximum theoretical specific gravity of the mix (Gmm) was determined following AASHTO T 209. Gmm was determined for two separate samples of the AC base for each project. In a few cases, fourteen cores were not available for the project site, therefore, Gmm was determined for two unconditioned (dry) cores used to determine indirect tensile strength as part of moisture damage testing. These cores were selected due to the very limited conditioning (77°F (25°C) for 2 hours) required. Due to the gradation envelopes for ODOT Item 301 and Item 302 AC bases and AASHTO T 209 specification, the sample size required was 8.8 lb (4 kg). Initially, a 4.4 lb (2 kg) capacity pycnometer was utilized, therefore each sample was split into two 4.4 lb (2 kg) samples and tested separately, requiring a total of four tests per project site. The resulting Gmm for each test was then averaged among the four, to represent Gmm for the project site. The research team later acquired an 8.8 lb (4 kg) capacity pycnometer, allowing for only one test per sample, and two tests per project site. In this case the two resultant Gmm values were averaged to obtain the Gmm for the project site. Testing was conducted in accordance with AASHTO T 209 with the exception the ODOT supplemental procedure for Gmm using saturated surface dry (SSD) outlined in ODOT Supplement 1036 (published October 21, 2016) was followed. Another notable deviation from AASHTO T 209 and ODOT Supplement 1036 was that samples were not cured or subjected to conditioning other than required to dry the sample when the initial Gmb was determined due to the fact the samples had been obtained from the field and were therefore, already cured.

Semi-Circular Bending (SCB) testing following the Illinois method, AASHTO TP 124-16 was conducted for a minimum of 6 specimens prepared from three cores for each project site. Specimens were conditioned for two hours at 77°F (25°C) prior to testing in an Instrotek Auto SCB machine.

Moisture susceptibility was evaluated following the AASHTO T 283-07 standard. A minimum of six cores from each project site were selected for the test, to provide three specimens in each subset: conditioned (wet) and unconditioned (dry). Samples were selected such that the average air void content of both subsets were similar. Prior to testing, samples were cut to a height of 3.7 ± 0.2 in (95 ± 5 mm), where possible based on the thickness of the top AC base lift. Specimens in the dry subset were placed in an environmental chamber at 77°F (25°C) for 2 hours prior to indirect tensile testing. As part of the conditioning process, conditioned samples must reach 70% to 80% saturation prior to being subjected to a minimum of 16 hours at -0.4°F (-18°C), followed by 24 hours in a 140°F (60°C) water bath and 2 hours in a 77°F (25°C) water bath. In some cases, the saturation requirement could not be met. Where possible, back-up specimens were prepared and conditioned to replace the initial specimen. While in most cases saturation was reached for the back-up specimen, there were a few cases in which saturation was not reached. The average IDT strength for the conditioned specimens was determined using only specimens which met the saturation requirement. Once the IDT strength was determined, the specimen was pulled open and the interior was inspected for signs of stripping and broken aggregate. The tensile strength ratio (TSR) was determined as the ratio of the average IDT strength for the conditioned specimens to the average IDT strength for the unconditioned specimens. Initially, indirect tensile (IDT) strengths were determined for both the dry and wet specimens using the MTS machine. Due to a problem with the machine, testing was then conducted on the Auto_SCB machine by using the appropriate jig and an 11.2 kip (50 kN) load cell. Results for the testing are presented in Appendix K. The analysis excludes samples in which the dry subset were tested in the MTS and the wet subset tested in the Auto_SCB machine.

Cantabro testing was conducted following Tex-245-F (2014). This standard was selected because only two specimens are required for the test; the research team made an effort to minimize the number of cores from each site to reduce the impact on in-service roadways while also obtaining a sufficient sample size to conduct all of the proposed tests. For six of the 51 project sites, coring had been completed prior to the approval of the sampling and testing plan, as a result not enough cores were obtained to conduct Cantabro testing for those sites. Core specimens of the top AC base lift were cut, where the lift thickness allowed, to the specified height of 4.5 ± 0.2 in. (115 ± 5 mm). Testing deviated from the Tex-245-F standard in that the specimens were not conditioned to 77°F (25°C) prior to testing. Due to the location of the LA Abrasion test machine, the test temperature of the 77°F (25°C) could not be maintained for the specimens or within the drum. However, test temperatures of the specimens were recorded and ranged from 69°F (20.4°C) to 77°F (25.0°C).

Appendix I: Data Analysis and Summary of Results

In-Place Air-Voids

The importance of in-place air voids has been well documented, and the general consensus is compaction, or in-place air voids, is one of the most important factor affecting performance. Given the importance of in-place air voids, air voids were determined for the top AC base lift from each core sample. Results for each project site, including the average, minimum, maximum, standard deviation and coefficient of variation of the in-place air voids are presented in Table 11–17.

Table 11–17. Summary of in-place air voids.

Project ID	Plan Base Type	Mix Type Used	Number of Cores	Average In-Place Air Voids	Minimum In-Place Air Voids	Maximum In-Place Air Voids	Standard Deviation	Coefficient of Variance
ASD-39-0.00	302, APP	302	17	9.28%	7.73%	10.93%	0.88%	9.45%
ATH-33-11.74	302	302	16	2.44%	1.13%	3.43%	0.77%	31.48%
BUT-75-5.91	302	302	14	5.14%	4.27%	6.43%	0.73%	14.20%
CLA-70-13.98	302	302	12	4.13%	2.21%	7.24%	1.30%	31.62%
CLI-73-12.03	880	302	14	5.56%	4.26%	7.05%	0.88%	15.75%
CLI-73-6.52	880	301	13	8.43%	5.12%	14.40%	2.30%	27.21%
COS-36-20.83	302	302	14	7.06%	6.59%	7.56%	0.34%	4.83%
CUY/SUM-77-0.00/32.73	302	302	14	3.31%	1.62%	4.79%	1.03%	30.99%
CUY-71-0.00	302	302	14	8.04%	6.17%	10.35%	1.17%	14.51%
DEF-24-7.96	880	302	16	7.56%	6.88%	9.12%	0.58%	7.70%
DEL-23-17.64	302	302	14	5.75%	3.84%	8.04%	1.23%	21.48%
DEL-23-19.24	302	302	14	8.56%	5.84%	10.64%	1.16%	13.55%
FAI-33-13.25	880	301	14	7.23%	5.37%	11.42%	1.46%	20.20%
FAI-33-17.44	880	302	14	5.52%	4.06%	7.01%	0.94%	16.98%
FAI-33-17.44 SR 13	301	301	15	4.95%	3.45%	6.91%	1.00%	20.10%
FAI-33-19.79	880	302	14	5.87%	2.87%	7.78%	1.61%	27.35%
FAI-33-19.79 SR 18	301	301	14	8.15%	7.02%	10.43%	0.96%	11.79%
FAI-33-7.31	880	302	14	9.58%	5.77%	12.09%	1.74%	18.11%
FAY-35-2.57	880	UNK	16	6.56%	3.22%	10.04%	2.17%	33.12%
FRA/LIC-161-23.20/0.00	880	301	15	8.81%	5.40%	10.44%	1.43%	16.22%
FRA-270-21.67	302	302	14	2.87%	1.20%	4.79%	1.09%	38.20%
FRA-270-35.41 Part 2	302	302	14	5.34%	3.06%	7.16%	1.14%	21.38%
FRA-70-3.41	302	302	15	2.86%	1.36%	5.13%	1.06%	37.15%
FRA-71-5.29	302	302	14	2.76%	1.48%	4.37%	1.04%	37.80%
FUL-20-10.86	302	302	16	5.33%	3.66%	6.75%	0.97%	18.16%
GAL-35-8.32	301	301	16	6.94%	5.59%	8.00%	0.69%	9.96%
GRE/FAY-35-26.20/0.00	301	UNK	14	5.84%	4.58%	7.28%	0.84%	14.44%
HAM-264-6.87	301/ 302	UNK	14	3.01%	2.01%	4.39%	0.75%	25.09%
HAN/WOO-75-19.22/0.00	302	302	14	7.58%	6.19%	9.31%	1.19%	15.71%

Project ID	Plan Base Type	Mix Type Used	Number of Cores	Average In-Place Air Voids	Minimum In-Place Air Voids	Maximum In-Place Air Voids	Standard Deviation	Coefficient of Variance
HAN-30-3.00	880	302	14	5.84%	4.43%	8.39%	1.20%	20.53%
LAK-2-7.60	302	302	20	5.94%	3.27%	7.64%	1.18%	19.85%
LIC-161-1.83	301	301	14	4.03%	1.94%	5.57%	1.00%	24.71%
LUC-2-21.24	301	302	14	6.50%	5.40%	7.39%	0.65%	10.04%
LUC-75-6.70	302	302	14	7.53%	6.14%	9.38%	1.06%	14.06%
MAD-142-0.49	301	301	14	5.03%	2.08%	6.61%	1.10%	21.84%
MED-42-17.68	301	UNK	14	4.53%	2.18%	10.04%	2.13%	46.96%
MED-71-9.56	302	UNK	14	8.59%	6.98%	10.41%	1.03%	12.02%
MER-219-14.04 "301"	301/ 302	301	15	7.54%	4.53%	9.38%	1.44%	19.09%
MER-219-14.04 "302"	301/ 302	302	16	5.92%	3.13%	8.80%	1.60%	27.05%
MRW-71-3.17	302	UNK	12	4.66%	2.46%	6.26%	1.16%	24.92%
ROS-207-0.00	880	301	14	4.57%	2.47%	7.20%	1.29%	28.28%
SUM-77-17.20	880	302	14	4.04%	2.20%	5.47%	0.91%	22.46%
VIN-50-11.75	301	301	14	5.66%	4.62%	6.15%	0.44%	7.72%
WAR-71-14.20	880	302	14	6.04%	4.56%	8.05%	1.18%	19.52%
WAR-71-3.78	302	UNK	14	3.10%	1.12%	5.50%	0.98%	31.45%
WAY-30-11.86 "301"	301, APP	301	9	10.20%	9.11%	11.51%	0.66%	6.45%
WAY-30-11.86 "302"	302	302	10	5.12%	3.18%	7.33%	1.22%	23.82%
WOO-75-10.61	302	302	14	6.92%	5.18%	8.60%	1.16%	16.79%
WOO-75-19.43	301	UNK	16	6.32%	4.53%	8.52%	1.15%	18.24%
WOO-75-2.37	302	302	15	5.40%	3.85%	7.18%	0.82%	15.21%
WOO-795-2.01	302	302	14	4.48%	3.63%	5.62%	0.66%	14.67%

The average, minimum and maximum in-place air voids for each project are plotted in Figure 11-16, in increasing order of average in-place air voids. The error bars indicate the maximum and minimum in-place air voids from each project. As shown in the plot, several project sites had a very large spread which is a sign of variability. Historically, 7% in-place air voids has been considered an achievable value in the field, and earlier research established that for each 1% increase in air voids above 7% there is about a 10% loss in pavement life [Linden et al., 1989]. Although, air voids lower than this threshold may be desired in the AC base layer to improve fatigue resistance, given that ODOT and most states do not have an acceptance limit on air voids for AC base layers, 7% serves as a reasonable initial value to evaluate in-place air voids. For reference, this threshold is also shown in Figure 11-16. Of the 51 project sites sampled, 15 (29%) had average in-place air voids for the tested AC base lift greater than 7%. Of those 15 project sites, the mix type for six was ODOT Item 301, the mix type for eight was ODOT Item 302 and for one, the type used was unknown based on the JMFs approved for the project.

While the average air voids is important, the variability is also of importance. Variability was also evaluated based on the standard deviation at a project site. Of the 15 project sites with average in-place air voids greater than 7%, five had standard deviations less than 1%, indicating that the in-place air voids at that site were fairly consistent, albeit higher than desired. The remaining ten sites had standard deviations ranging from 1.03% to 2.30% indicating a wider spread and greater variability at the site, and high in-place air voids. For those sites that had an average in-place air voids less than 7%, 16 sites had a standard deviation less than 1%. The remaining sites, which had standard deviations greater than 1% had reasonable air voids, but greater variability.

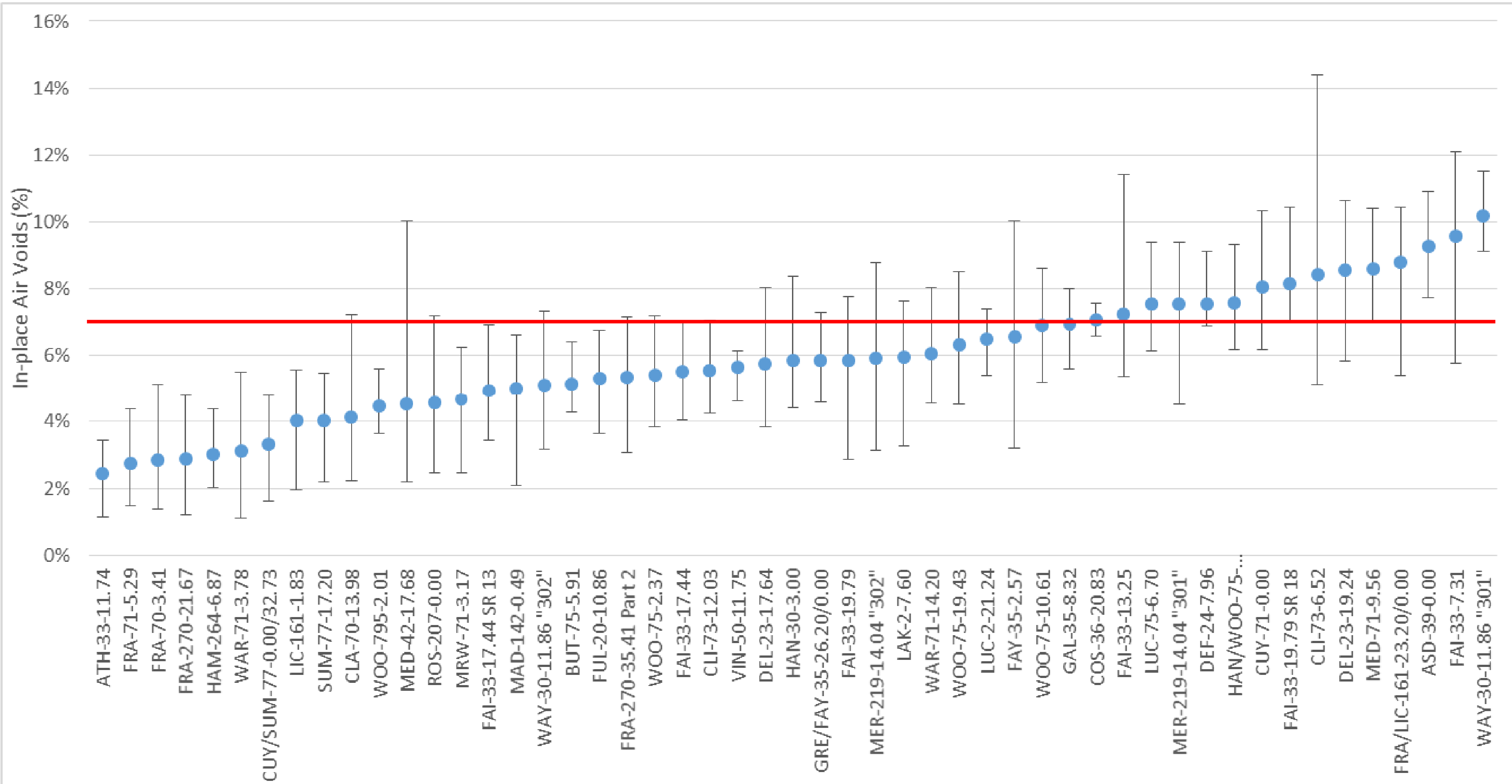


Figure 11-16. In-place air voids by project.

ODOT Item 880 serves as a warranty specification, therefore, there was interest in seeing if any effect on in-place air voids was evident. In-place air voids were summarized by the plan base type, as shown in Table 11–18. There were five projects which were designated in the plans as Item 301 as per plan (APP), Item 302 APP, or one lift was designated as Item 301 and the other as Item 302, shown here as 301/302. On average, in-place air voids for Item 880 AC bases were the highest, at 6.61% in-place air voids. However, it should be noted the average was less than 7% for each plan base type with the exception of the two "as per plan" plan base types. However, all had large standard deviations, indicating high variability, making it difficult to draw conclusions regarding the effect of ODOT Item 880 on in-place air voids.

Table 11–18. Summary of in-place air voids by plan base type.

Plan Base Type	Number of Project Sites	Number of Cores	Average In-place Air Voids	Standard Deviation	Coefficient of Variation
301	10	145	5.81%	1.57%	26.97%
302	23	326	5.33%	2.16%	40.44%
880	13	186	6.61%	2.11%	31.95%
301/302	3	45	5.44%	2.29%	42.03%
301, APP	1	9	10.20%	N/A	N/A
302, APP	1	17	9.28%	N/A	N/A

It should be kept in mind that the project sites included in this study ranged in age from brand new or under construction to 20 years old. Additionally, over that time changes have been made to ODOT CMS that influenced the in-place air voids. Prior to 2002, the design air content for Item 302 was 5%. Design air content was reduced from 5% to 4.5% with the 2002 specifications and to 4% with the 2008 specifications. Cumulative frequency plots were developed to take into account the base type (ODOT Item 301, 302, or 880) as well as the specification year for which the plans were held to. These changes in the ODOT CMS are reflected in the cumulative frequency plots for ODOT Item 301, ODOT Item 302, and ODOT SS 880, presented in Figure 11-17, Figure 11-18, and Figure 11-19, respectively.

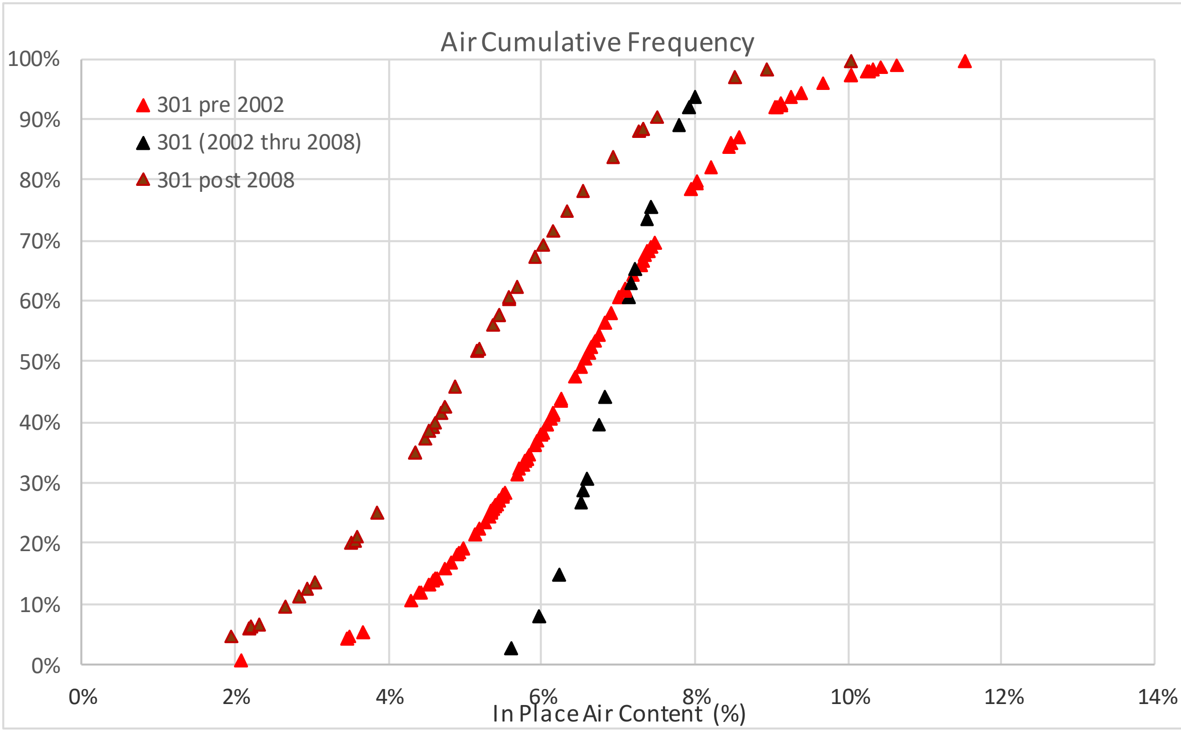


Figure 11-17. Cumulative frequency for in-place air voids for ODOT Item 301.

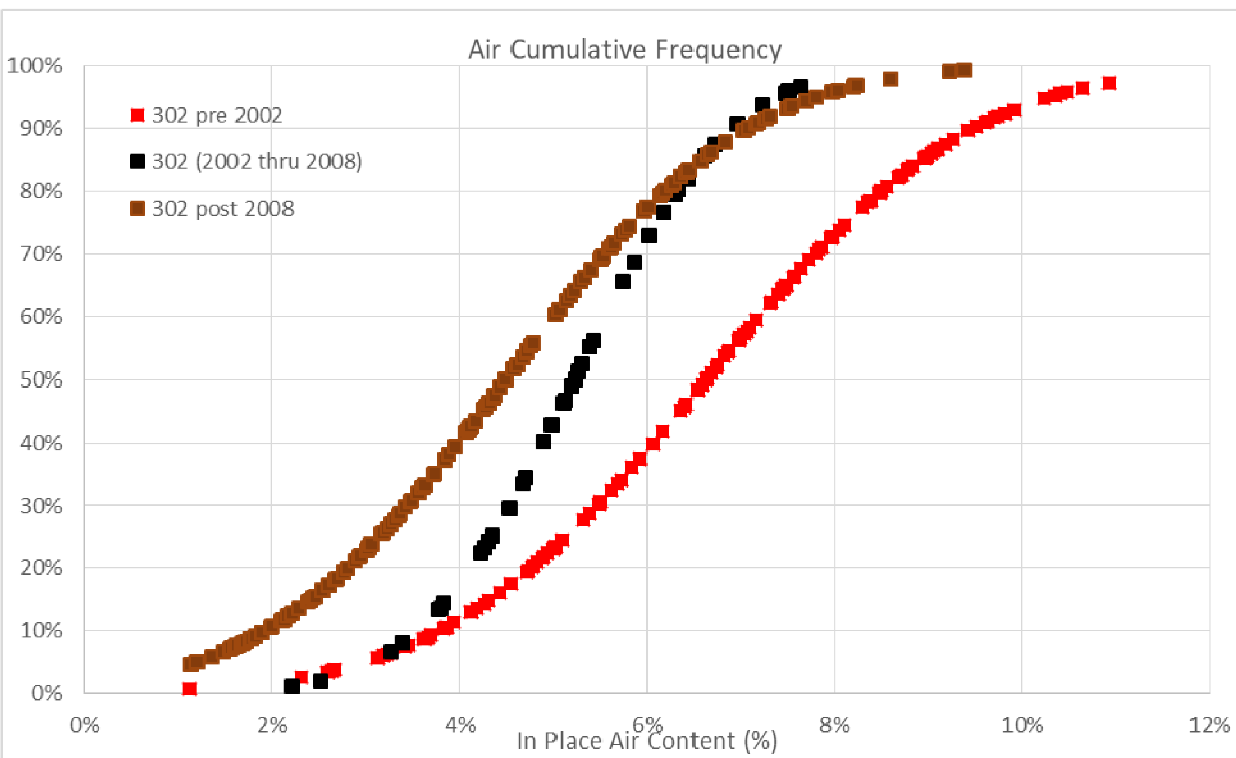


Figure 11-18. Cumulative frequency for in-place air voids for ODOT Item 302.

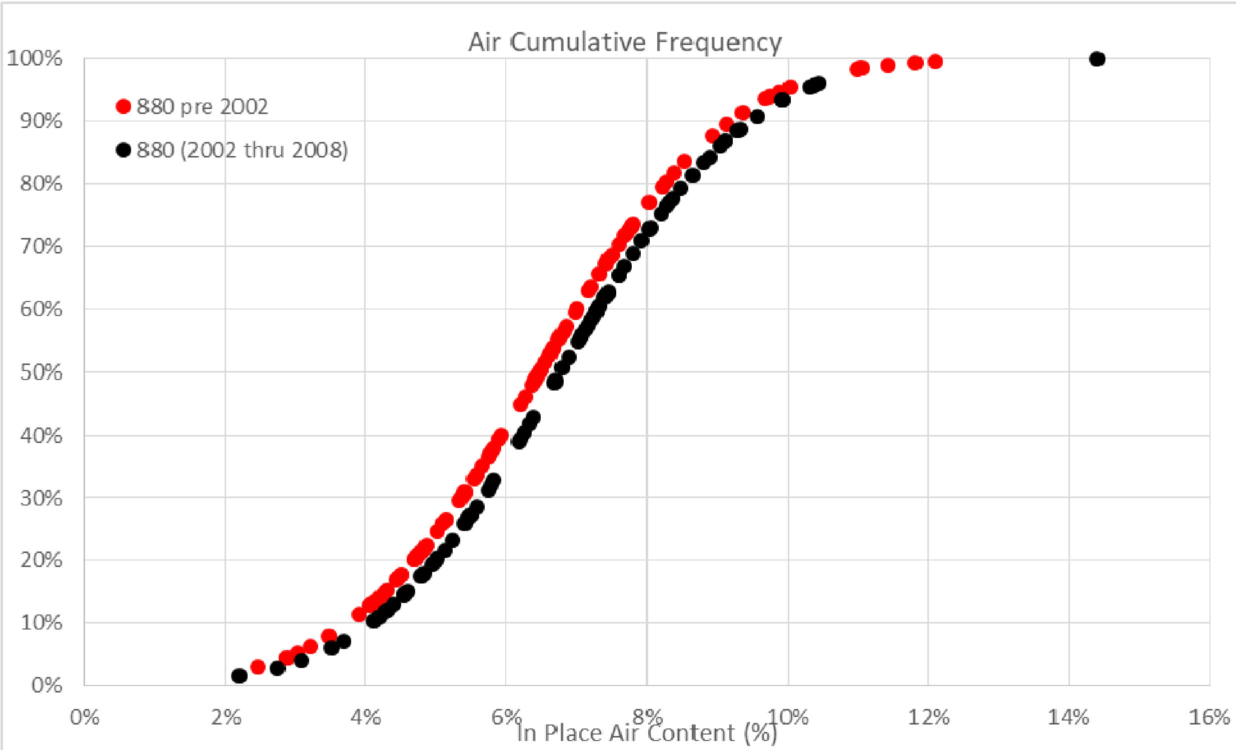


Figure 11-19. Cumulative frequency for in-place air voids for ODOT SS 880.

Lastly, in-place air voids were evaluated relative to the performance category based on field performance measured by historical PCR for each project site. Shown in Figure 11-20, below the average in-place air voids along with the minimum and maximum, represented by the error bars, are shown for each project within the performance categories. While there are no strong trends evident, all projects performing exceptionally had an average in-place air voids of approximately 8% or less, and two of the projects with the highest average in-place air voids fell into the poor performance category. The average and standard deviation of the in-place air voids from core samples are provided in Table 11–19 for each performance category. On average, pavements with poor field performance had the highest in-place air voids and exceptional pavements had the lowest in-place air voids. However, the AC base layer is deep in the structure and therefore effects on performance due to the quality of the AC base may take longer to trigger distress at the surface, and PCR is based solely on distresses observed at the surface.

Table 11–19. Summary of In-place Air Voids by Performance Category

Performance	Number of Projects	Number of Cores	Average In-place Air Voids	Standard Deviation
Exceptional	5	70	5.80%	1.73%
Average	16	233	6.19%	1.98%
Poor	9	127	6.74%	2.15%
N/A	21	298	5.38%	2.33%

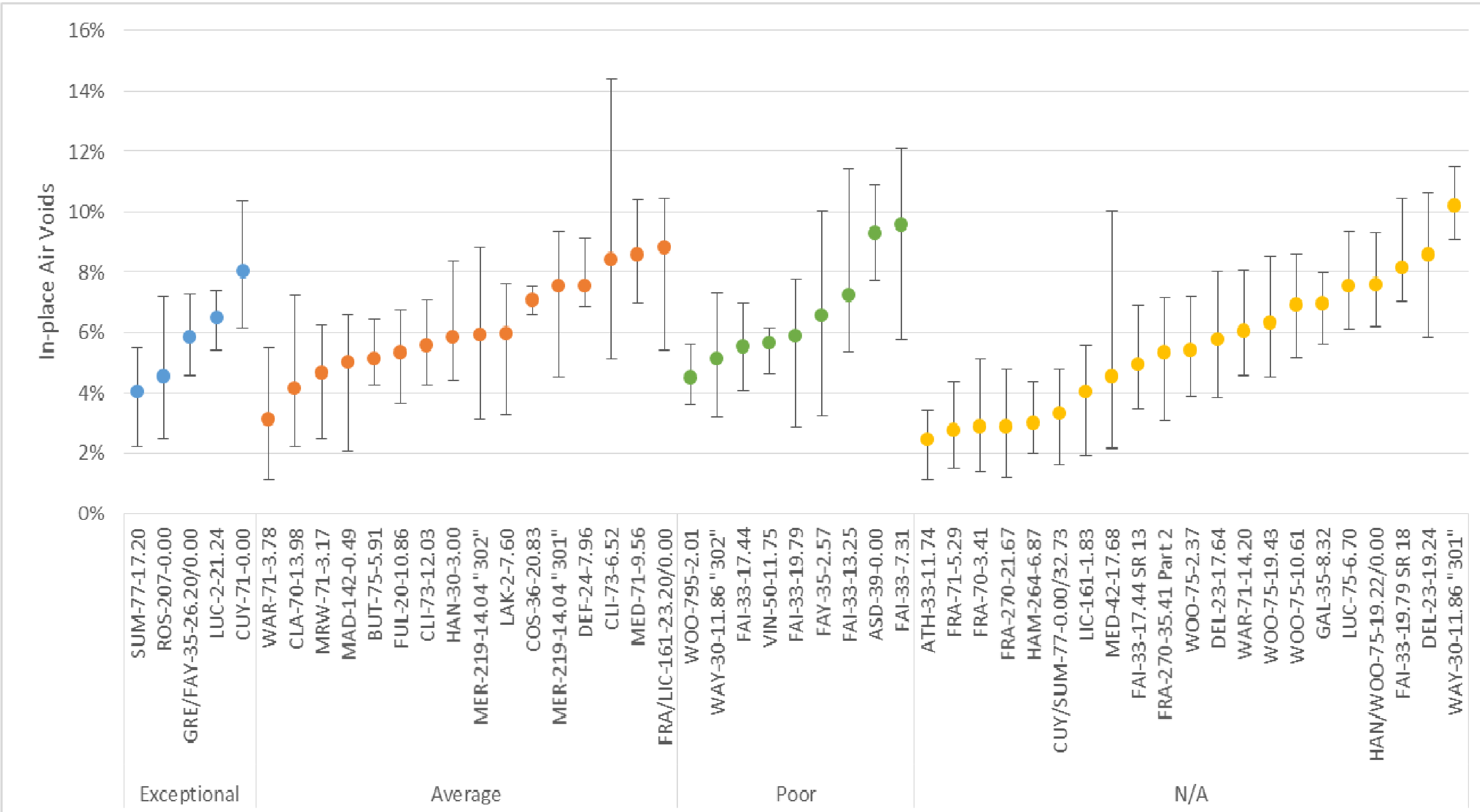


Figure 11-20. In-place air voids by performance category.

SCB-IL

To evaluate the cracking resistance of the in-place AC base mixes SCB testing following the Illinois method (SCB-IL) was conducted. A summary of results by project are provided in Appendix X along with full results. First, flexibility index (FI) was plotted against specimen air voids for all specimens tested. As shown in Figure 11-21, there is a slight downward trend in FI as specimen air voids increase, however, the relationship is very poor. On the other hand, when fracture energy is plotted against specimen air voids, there is a notable downward trend with increasing air voids, although there is still significant spread and a relatively low R^2, as shown in Figure 11-22.

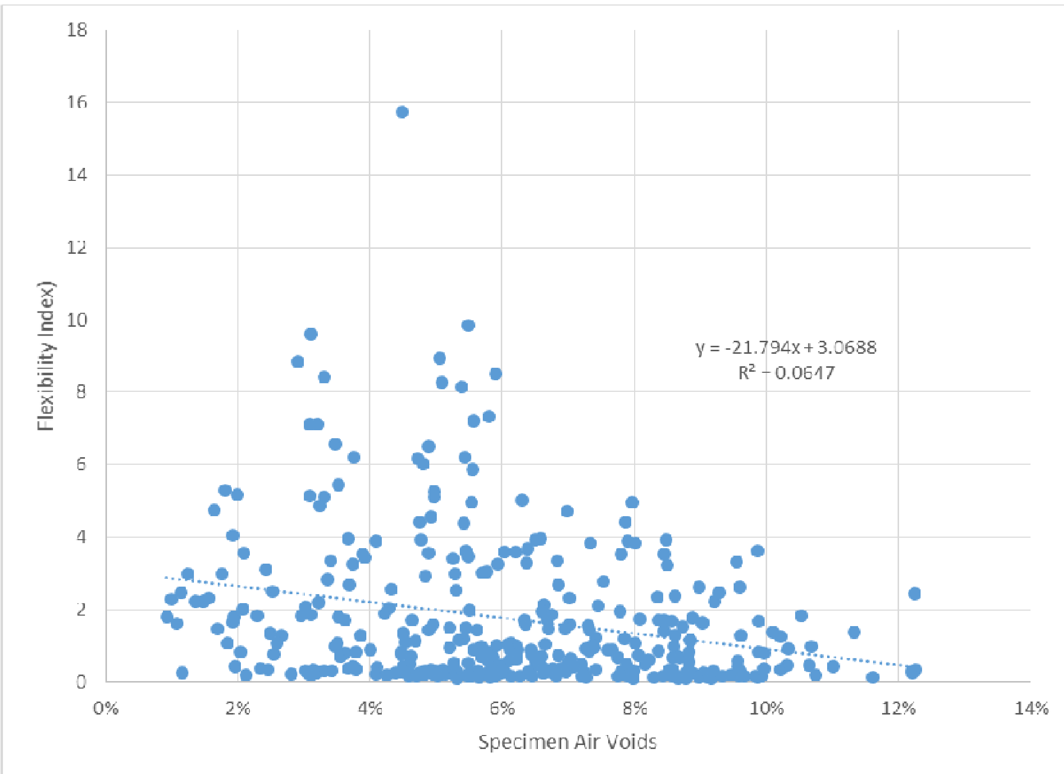


Figure 11-21. Flexibility Index vs. Specimen Air Voids.

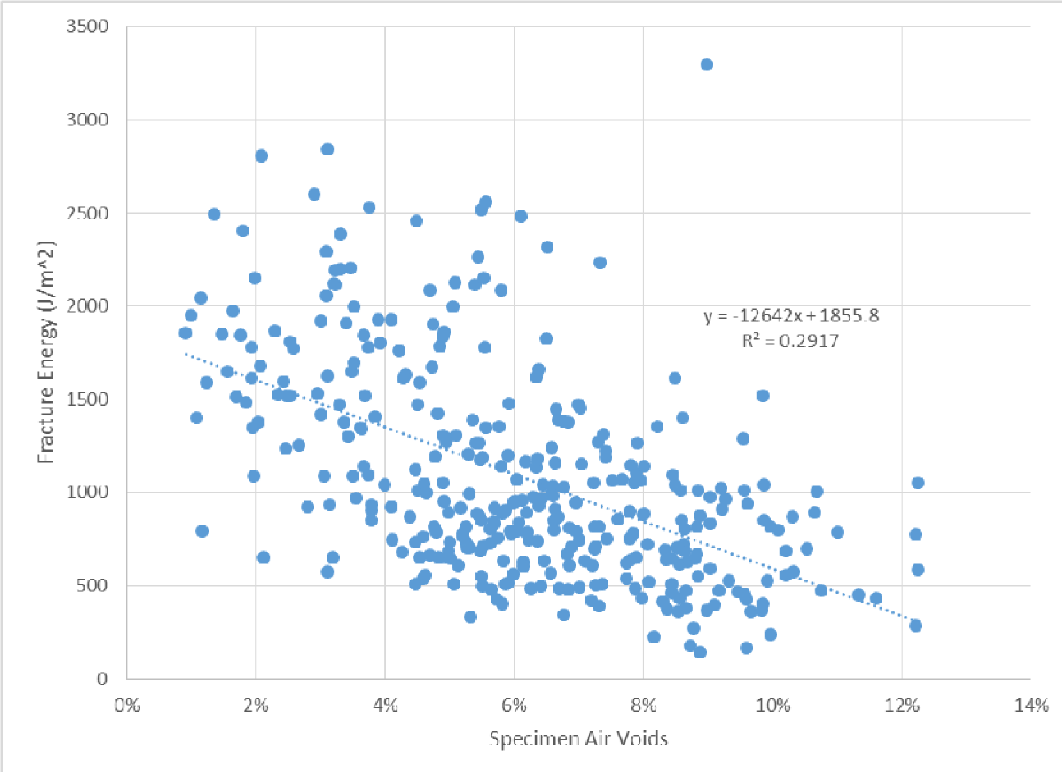


Figure 11-22. Fracture Energy vs. Specimen Air Voids (1000 J/m^2 = 0.088 BTU/ft^2).

Although fracture energy is indicative of crack resistance, Al-Qadi et al. [2015] found FI to be a better indicator of brittle mixes. Therefore focus was placed on FI. In the development of the SCB-IL method and FI, Al-Qadi et al. [2015] found for field specimens of the wearing course taken from recently constructed pavements in Illinois, good performing sections had FI of 10 or greater, while poor performing sections had FI of 6 or less. Based on performance for AC mixes tested in FHWA's ALF, Al-Qadi et al. [2015] also reported an FI of 2.0 appears to be a cut-off value for poor performing sections while sections with FI greater than 6.0 were in the good-performing category. The SCB-IL method and FI were developed for AC mixes with NMAS of 19.0 mm (0.75 in) and less, therefore the values Al-Qadi et al. [2015] found to distinguish between good and poor performance in the field for new AC mixes may not be applicable for new AC base mixes in Ohio due to the coarser gradation of ODOT base mixes. To date, no thresholds exist for older pavements, or for AC mixes with larger NMAS, therefore, the initial FI values based on field performance established by Al-Qadi et al. [2015] maybe useful for a rough comparison.

Plotted in Figure 11-23 are the average FI for each project, with the pavement age plotted on the secondary axis. Average FI ranged from 0.67 to 8.9 for new AC base mixes. As shown in Figure 11-23, the largest average FI for a project site was approximately 9, and the next highest was 6. Both of these values corresponds to project sites under construction at the time of coring, LIC-161-1.83 and MED-42-17.68, respectively. Despite being the two largest FI of all 51 project sites, neither would be classified as good performing sections by the initial thresholds identified by Al-Qadi et al. [2015]. With the exception of LIC-161-1.83, all of the newly constructed AC bases have average FI less than 6 and would be classified as poor performing sections by the initial thresholds from Al-Qadi et al. [2015].

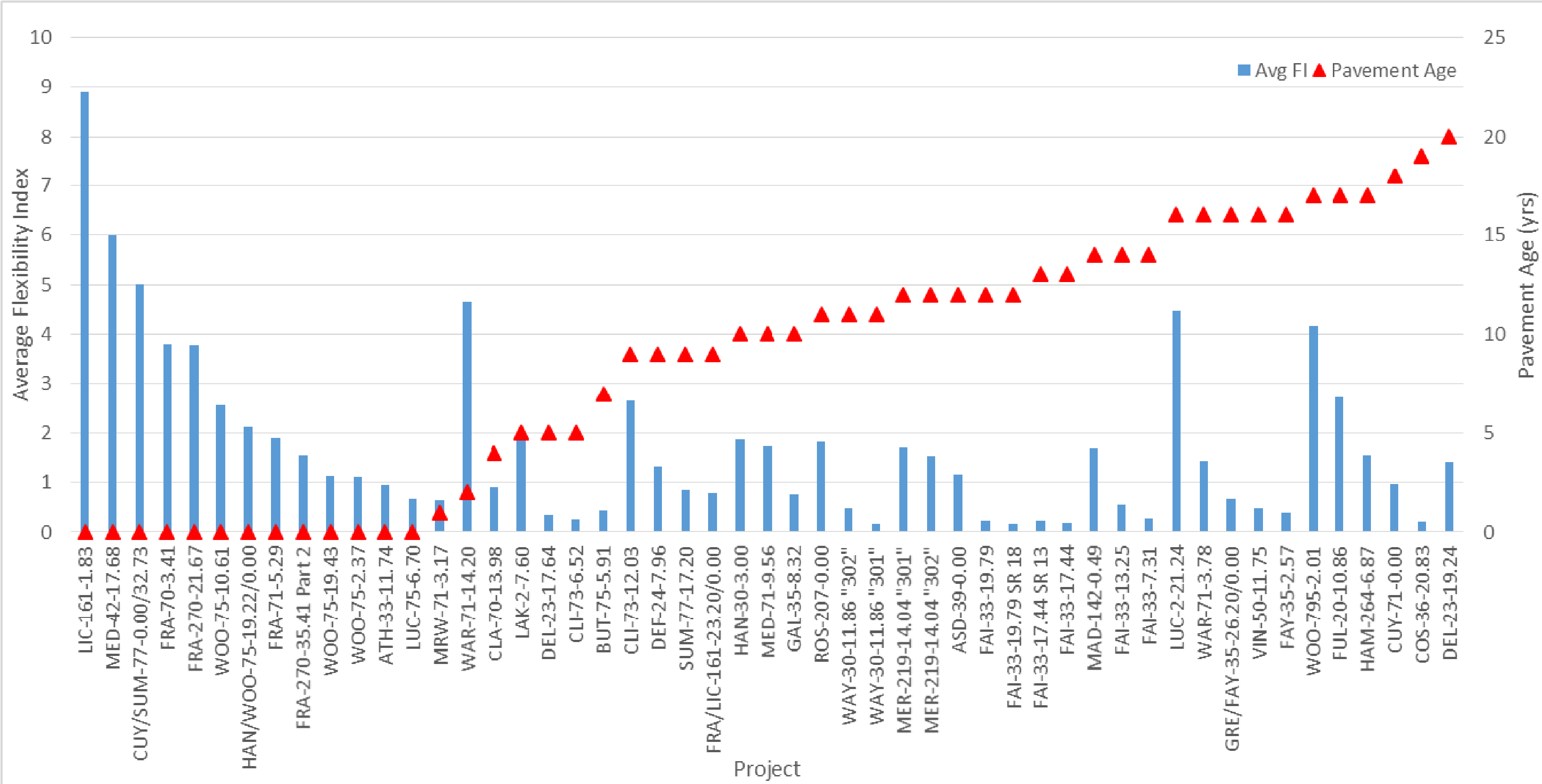


Figure 11-23. Average FI and pavement age for each project site.

Al-Qadi et al. [2015] found FI decreased with age. While that was generally the case for the project sites investigated here, there are several older AC bases with FI values as good as or better than new AC bases. The average FI and standard deviation of FI are summarized by age group based on all specimens tested in the age group, as shown in Table 11–20. Average FI generally decreases with each age group. Interestingly, the exception to this trend is the oldest age group, pavements that are 16 to 20 years old, which has the second highest FI, on average. This may be related to changes made in the ODOT Item 302 specifications. Multiple changes were made with each revision between 1997 and 2005 therefore, it is difficult to isolate the changes related to the higher FI. The lowest FI on average was for AC bases 11 to 15 years old. As shown later in Figure 11-25, several project sites had a wide range of FI. While the new AC bases had the highest average FI, that age group also had the largest standard deviation, indicating high variability. The standard deviations for the remaining age group were greater than the associated average FI, again indicating high variability.

Table 11–20. Summary of FI by Age Group

Age Group (yr)	Number of Projects	Number of Specimens	FI Average	FI Standard Deviation
0	13	87	3.12	2.76
1-5	6	48	1.46	2.07
6-10	8	54	1.24	1.16
11-15	13	90	0.84	1.04
16-20	11	70	1.75	1.82

Although variability is observed, it is difficult to determine the source of the variability, as two different base mixes were utilized, ODOT Item 301 and ODOT Item 302, and varying levels or recycled material (RAP, RAS, or RAP and RAS) were utilized, as well as the previously discussed variability associated with in-place air voids. The influence of the large aggregate and segregation on the fracture band may also induce variability. The two SCB specimens shown in Figure 11-24 were taken 379 ft (116 m) apart on the same project, FRA-71-5.29.

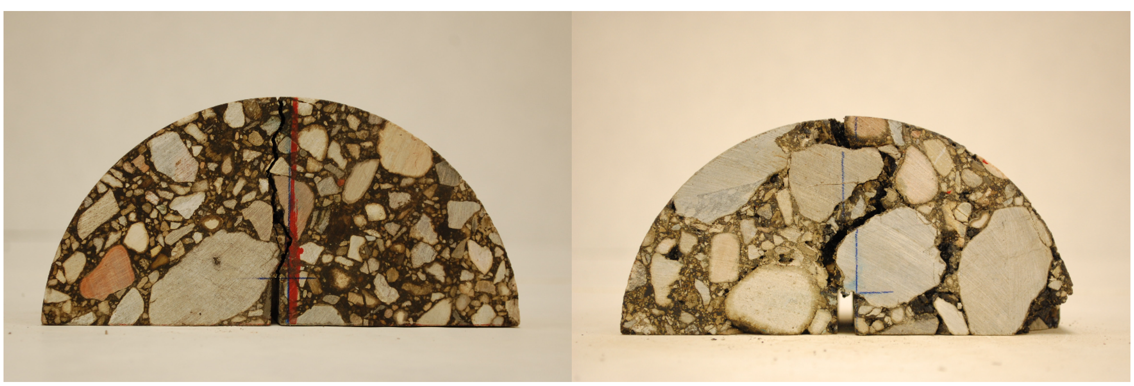


Figure 11-24. SCB specimens from FRA-71-5.29.

The average FI for each project site along with the minimum and maximum FI (shown as error bars) are plotted by performance category in increasing order of FI in Figure 11-25 There are several projects in which PCR performance data were not available, marked “N/A” in the plot, due to the location from where the base material was sampled (shoulder, ramp, or service road (SR)), or because the project was under construction and has not yet received traffic. In looking at those project sites that fell into the exceptional performance category based on historical PCR data, only one had an FI greater than 2. However, pavement age for those project sites categorized as exceptional performance ranged from 9 to 18 years old, and therefore it is expected they would have FI values on the low side. Pavement ages for those project sites with poor performance were in a similar range: 11 to 17 years. Of these 9 project sites, 7 had an average FI less than 1.00.

The average FI for a project site was also evaluated based on the use of recycled material. Based on the review of the JMFs, project sites were categorized as "virgin" (no RAP or RAS used in the mix), "recycled" (RAP, RAS, or both RAP and RAS were used in the mix) or "unknown," in which the JMF could not be obtained, or multiple JMFs existed which included virgin or recycled mixes making it difficult to identify which mix was used within the sampling limits. Within those categories, they are further broken down to indicate what material (RAP or RAS, or both) was used based on the information identified in the JMFs. As shown in Figure 11-26, the average FI for a project site is plotted based on these categories, and subcategories. Project sites are plotted in increasing order of age within each subcategory. AC base mixes produced with RAP (and no RAS) represented the largest portion of the project sites evaluated, and also represent the highest and lowest FI. There is no clear trend for FI between AC base mixes produced with or without recycled material.

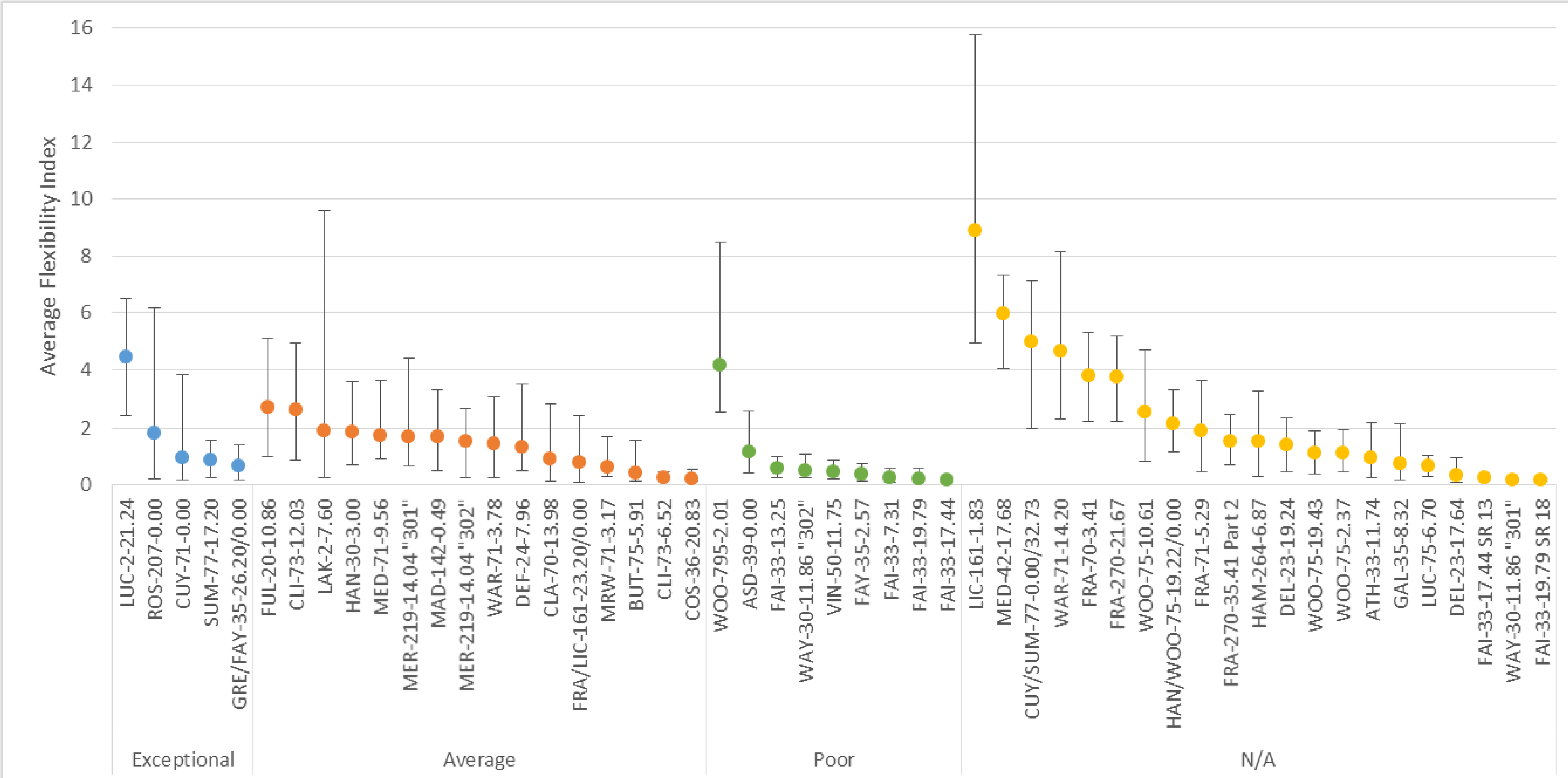


Figure 11-25. Average FI for each project site by performance category.

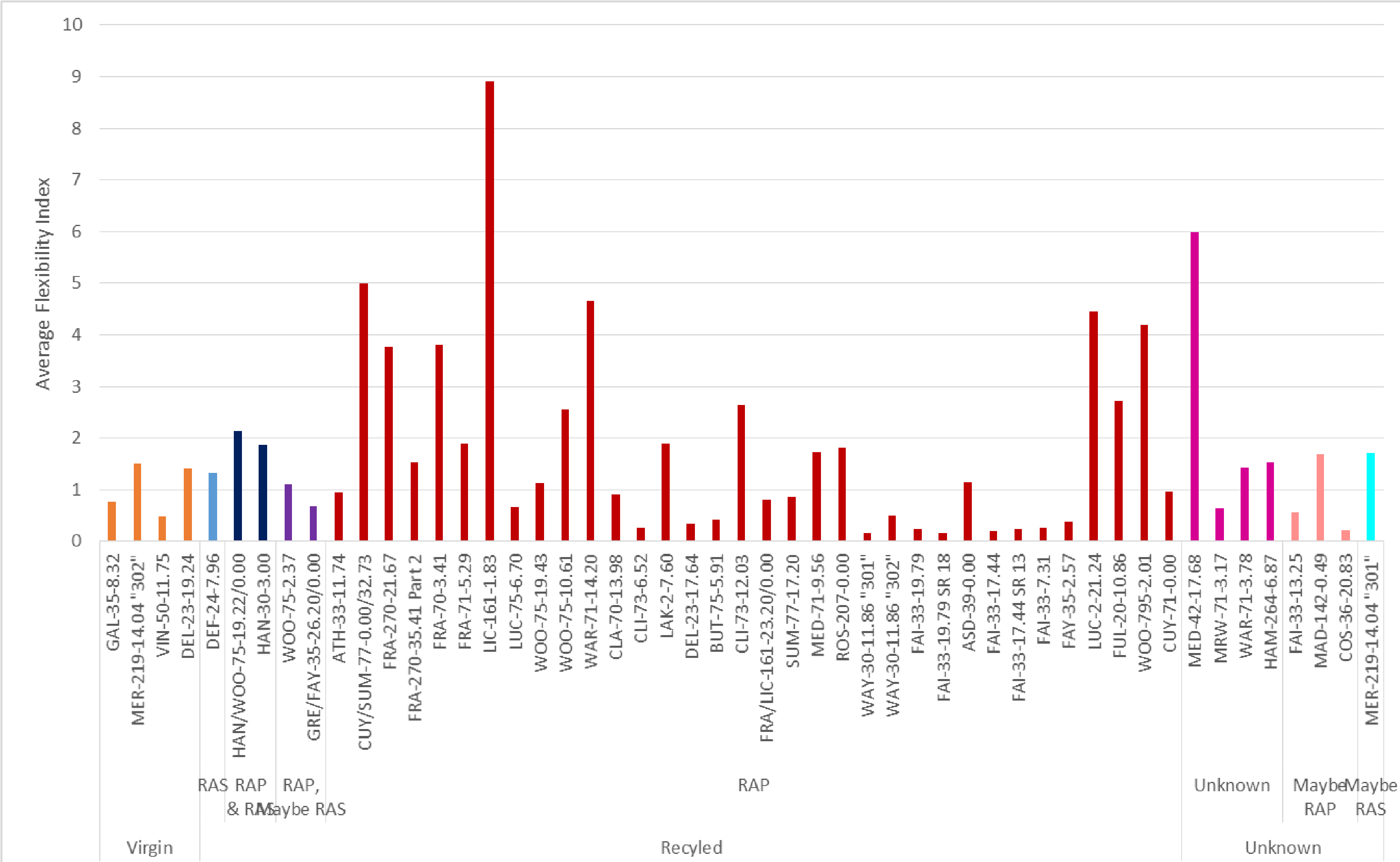


Figure 11-26. Average FI by project site and recycled material.

To investigate if aggregate type has any effect on flexibility index, the average FI for each project site was plotted in Figure 11-27 based on the aggregate type used in the mix, as determined from the review of project JMFs. The age of the pavements are shown on the plot above each bar for each project site. It appears that gravel mixes tended to have lower FI, however these samples were taken from pavements that ranged in age from 10 to 16 years old, and therefore it would be expected that they have lower FI. Projects using a blend of gravel and limestone had similar FI for pavements in the same age range (10 years or older) with three of the four pavements in that age range having FI less than 1.0. For the same age range limestone mixes generally had higher FI, although there were more limestone mixes than either gravel or blended aggregate mixes. Of the 14 limestone mixes 10 years or older, only 6 had average FI less than 1.0, and of the remaining 8, three were greater than 2.0, two of which were greater than 4.0.

The effect of mix type (ODOT Item 301 or 302) on FI was also investigated. Shown in Figure 11-28, are average FI values for each project, separated by mix type, and labeled with pavement age above each bar in the plot. The highest average FI (8.90) and lowest average FI (0.15) among all projects were represented by Item 301 mixes. However only one of the 12 projects where Item 301 was placed was a new pavement. The majority (9) of projects where Item 301 were 10 years or older. Whereas approximately one third of the projects (10) with Item 302 were new pavements and 42% (13) were ten years or older. Therefore, only projects 10 years or older can be used to compare FI of Items 301 and 302 mixes. Summarized in Table 11-21 below, are the mean, minimum and maximum average FI for projects with pavements 10 years or older. On average Item 302 mixes had average FI 1.8 times greater than Item 301 mixes for older pavements.

Table 11–21 Summary of average FI by mix type for pavements 10 years or older

Mix Type Placed	No. of Projects	Mean FI	Minimum FI	Maximum FI
301	9	0.84	0.15	1.81
302	13	1.51	0.19	4.46

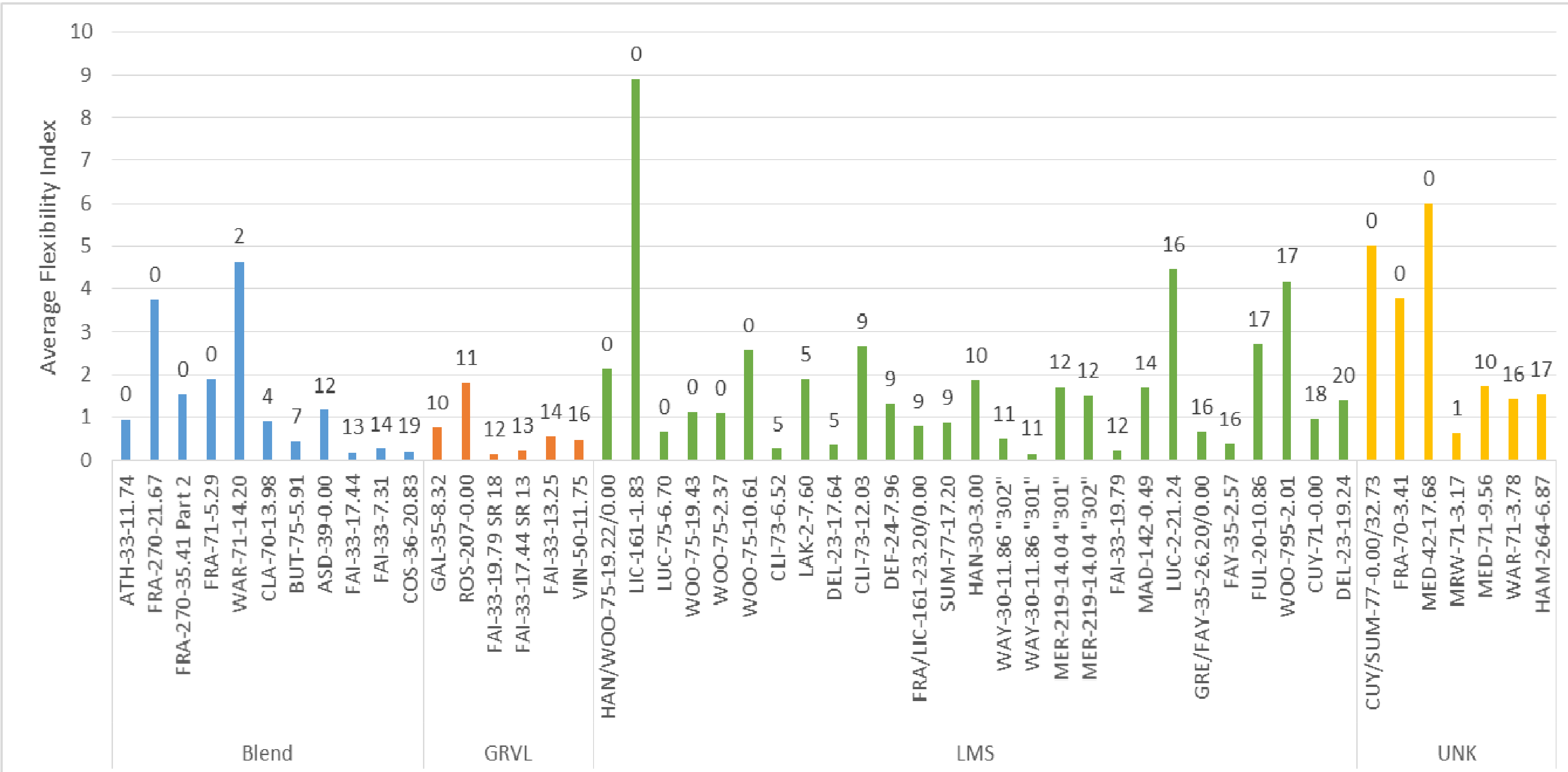


Figure 11-27. Average FI by project site and aggregate type.

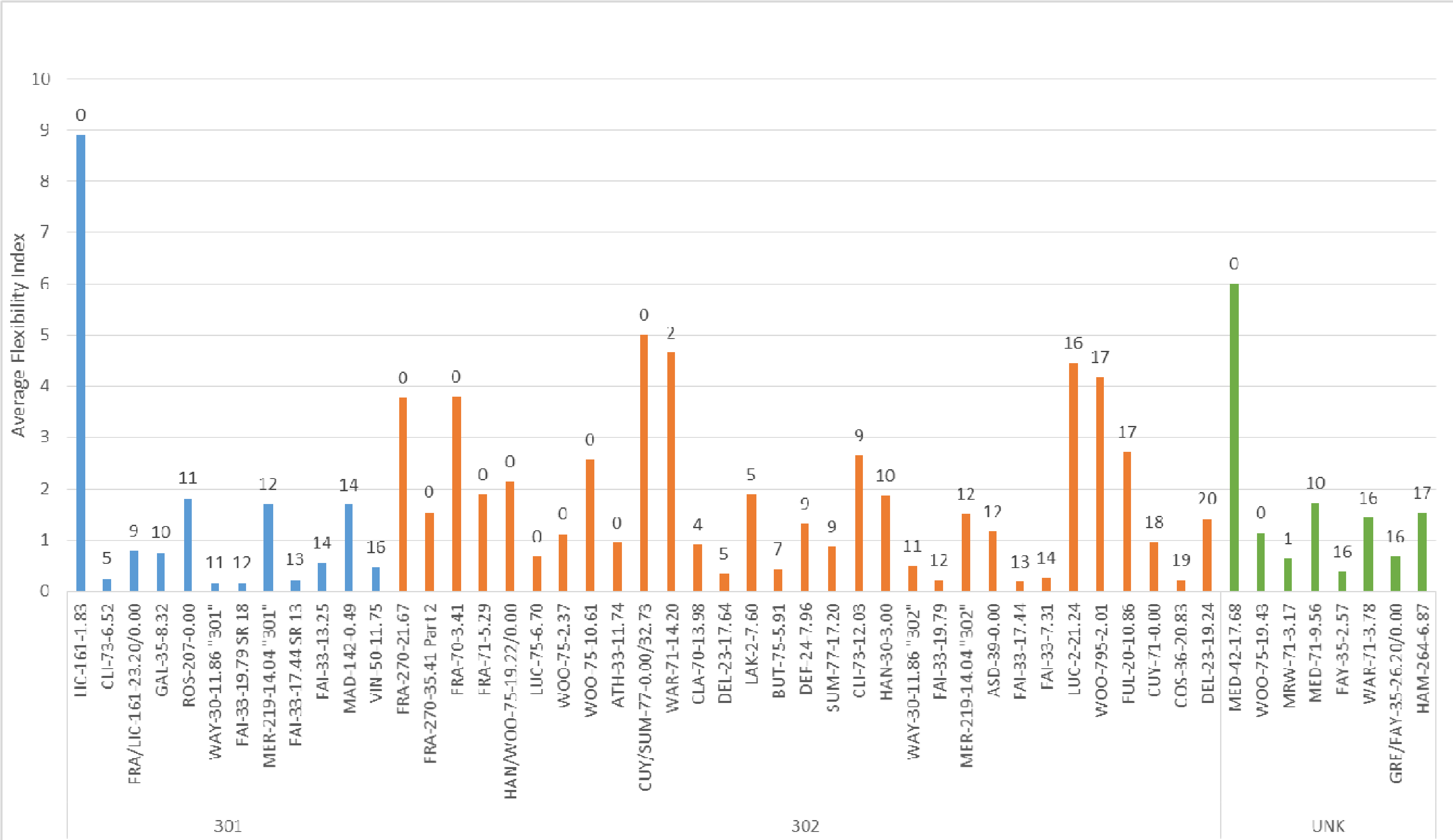


Figure 11-28. Average FI by mix type.

TSR

Indirect tensile strength testing was completed from which the tensile strength ratio (TSR) was determined as the ratio of average conditioned tensile strength to average unconditioned tensile strength. TSR values are used to assess the moisture susceptibility of the mix. Additionally, TSR is typically compared against an established criterion during the mix design to determine the need for anti-strip additives. If the criterion is not met, the mix is considered susceptible to moisture damage and therefore, requires an anti-strip additive. The TSR criterion for ODOT Item 302 is 0.70; there is no TSR criterion for ODOT Item 301.

Average tensile strength for the unconditioned (dry) and conditioned (wet) samples are summarized along with the TSR values for each project in Appendix K. Of the 43 AC base mixes evaluated, only 19 met the 0.70 criterion. Although ODOT has a criterion for TSR for Item 302 mixes, the tensile strength of the conditioned and unconditioned samples are also important to consider. For the 19 project sites with AC bases mixes meeting the TSR criterion, the average tensile strength for the unconditioned and conditioned samples were plotted, as shown in Figure 11-29. Also shown in the plot are the TSR values for each project site. For two of the 19 project sites, CLI-73-12.03 and LUC-2-21.24, the tensile strength for both conditioned and unconditioned samples were less than 100 psi (689 kPa), a value sometimes considered a minimum value when used in conjunction with a TSR criterion.

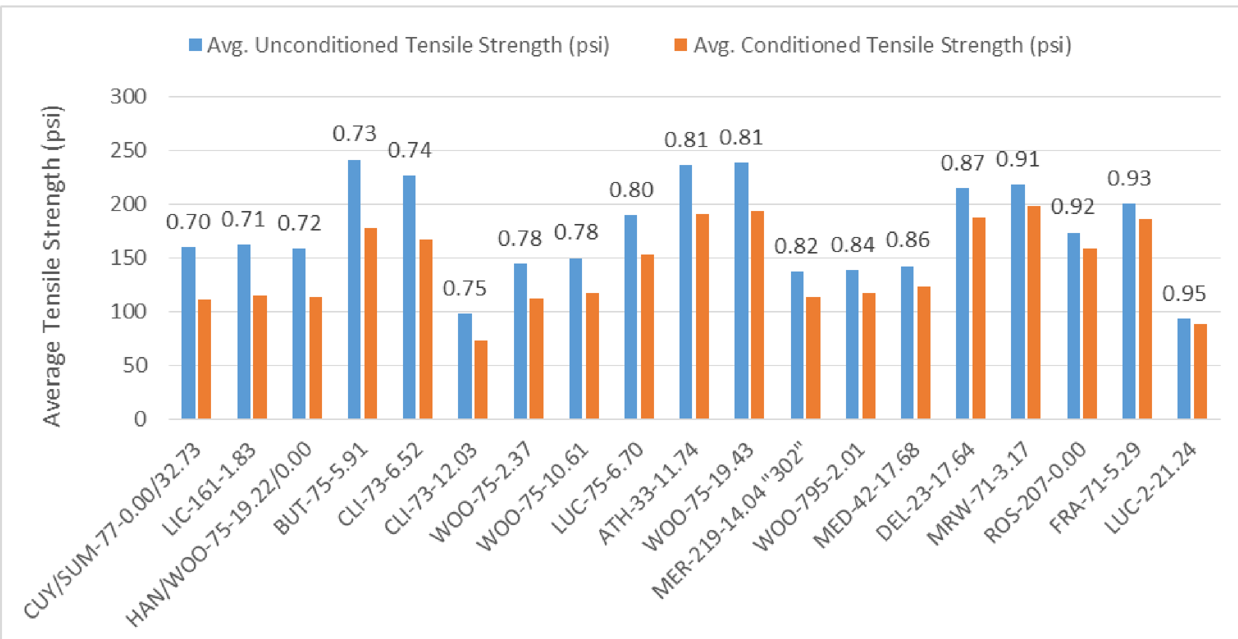


Figure 11-29. Tensile strength for project sites with TSR meeting 0.70 criterion.

To determine if there was any effect of using the SS 880, the number of projects meeting or exceeding this criteria were summarized in Table 11–22 by the plan base type. Based on this summary, the majority (75%) of AC bases in which SS 880 was specified in the plans failed to meet the TSR criterion.

Table 11–22. Summary of projects meeting TSR criterion by plan base type

Plan Base Type	Number of Projects	Average TSR	Number of Projects TSR ≥ 0.7	Percent of Projects TSR ≥ 0.7	Average Pavement Age (yr)
301	9	0.73	4	44%	9.2
302	18	0.70	11	61%	5.3
880	12	0.62	3	25%	10.9
301/302	3	0.70	1	33%	13.7
302, APP	1	0.53	N/A	N/A	12.0

TSR values were compared for the two mix types (Item 301 or Item 302) placed. For some project sites the mix type placed differs from the mix type specified in the plans. This is true where the plans indicate SS 880 as the JMF and therefore, the mix type placed will fall under either Item 301 or Item 302 (or unknown if it could not be determined from multiple JMFs). For one project site Item 301 was specified in the plans, however, Item 302 was placed. For several projects the mix type placed could not be determined because at least one JMF for each Item 301 and Item 302 were approved for the project. TSR values for each project site according to mix type are plotted in Figure 11-30. The TSR criterion of 0.70 is also shown for reference. The pavement age is labeled for each project site in the plot. With the exception of LIC-161-1.83, the majority of Item 301 AC base mixes tested had TSR values less than the criterion. As noted TSR is not a requirement for this mix. Given the requirement in the specification for Item 302, it is expected those bases mixes would meet this criterion. However, there are three sites, FRA-270-21.67, FRA-70-3.41, and FRA-270-35.41 Part 2, for which the Item 302 AC base was newly constructed and TSR values were less than 0.70. Interestingly, three project sites utilized the same JMF, including two of those aforementioned project sites: FRA-270-21.67, FRA-270-35.41 Part 2 and FRA-71-5.29.

Of the project sites which used Item 302 base mixes just under 50% met or exceeded the 0.70 criterion, whereas only 30% of the project sites with Item 301 base mix passed the criterion. Age may play a factor, as shown in Figure 11-30, the majority of Item 301 base mixes were older pavements. Although the projects selected for Item 302 mixes represented a larger variety of pavement ages, the majority of those newly constructed (with the exception of those noted earlier) passed the TSR criterion.

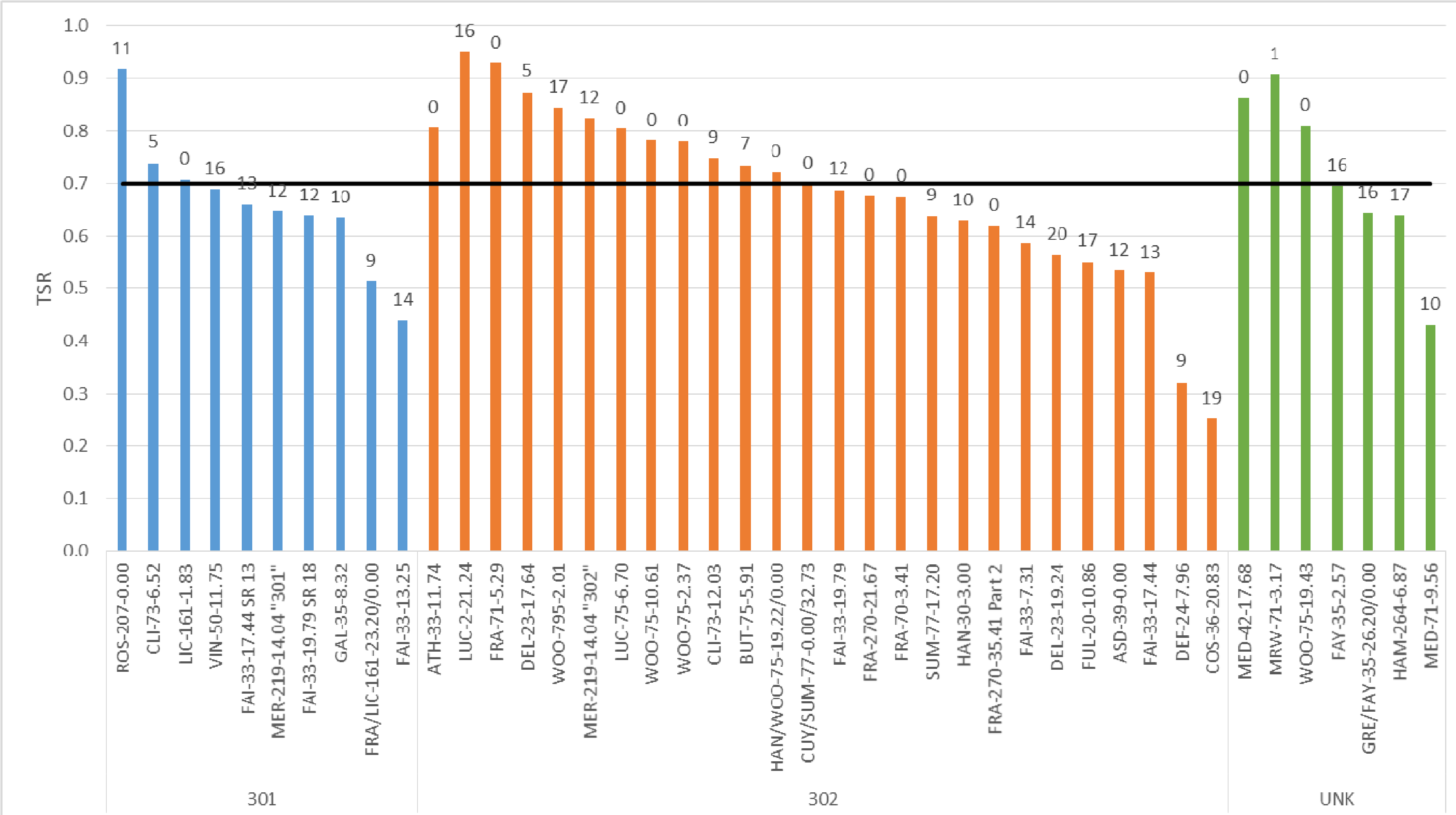


Figure 11-30. TSR values by mix type placed.

Many of the ODOT Item 301 mixes tested were in older pavements, therefore TSR was plotted against pavement age in Figure 11-31 to determine if there was any effect due to age. There is downward trend in TSR with increasing age, however the relationship is weak.

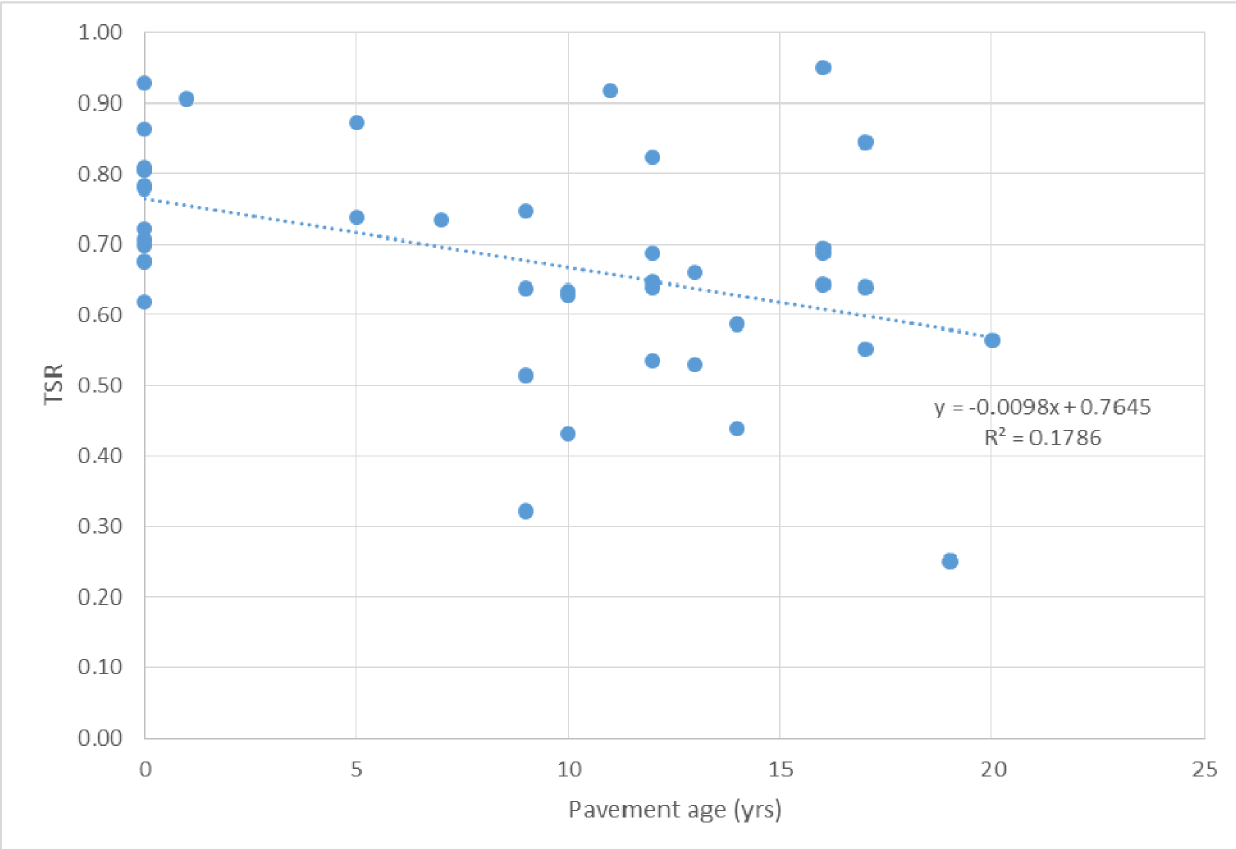


Figure 11-31. TSR vs. Pavement Age.

The relationship between field performance and TSR was evaluated by plotting the TSR values for each project site according to performance category, as shown in Figure 11-32. The pavement ages and TSR criterion are shown for clarity. There was a wide range of TSR values for the AC base mixes sampled from project sites with average field performance. This category included three project sites for which TSR values of the AC base mixes were the lowest TSR values of all those analyzed here. While two of the four pavements with exceptional field performance met the TSR criterion, one, LUC-2-21.24, had low unconditioned and conditioned tensile strength as shown above in Figure 11-29.

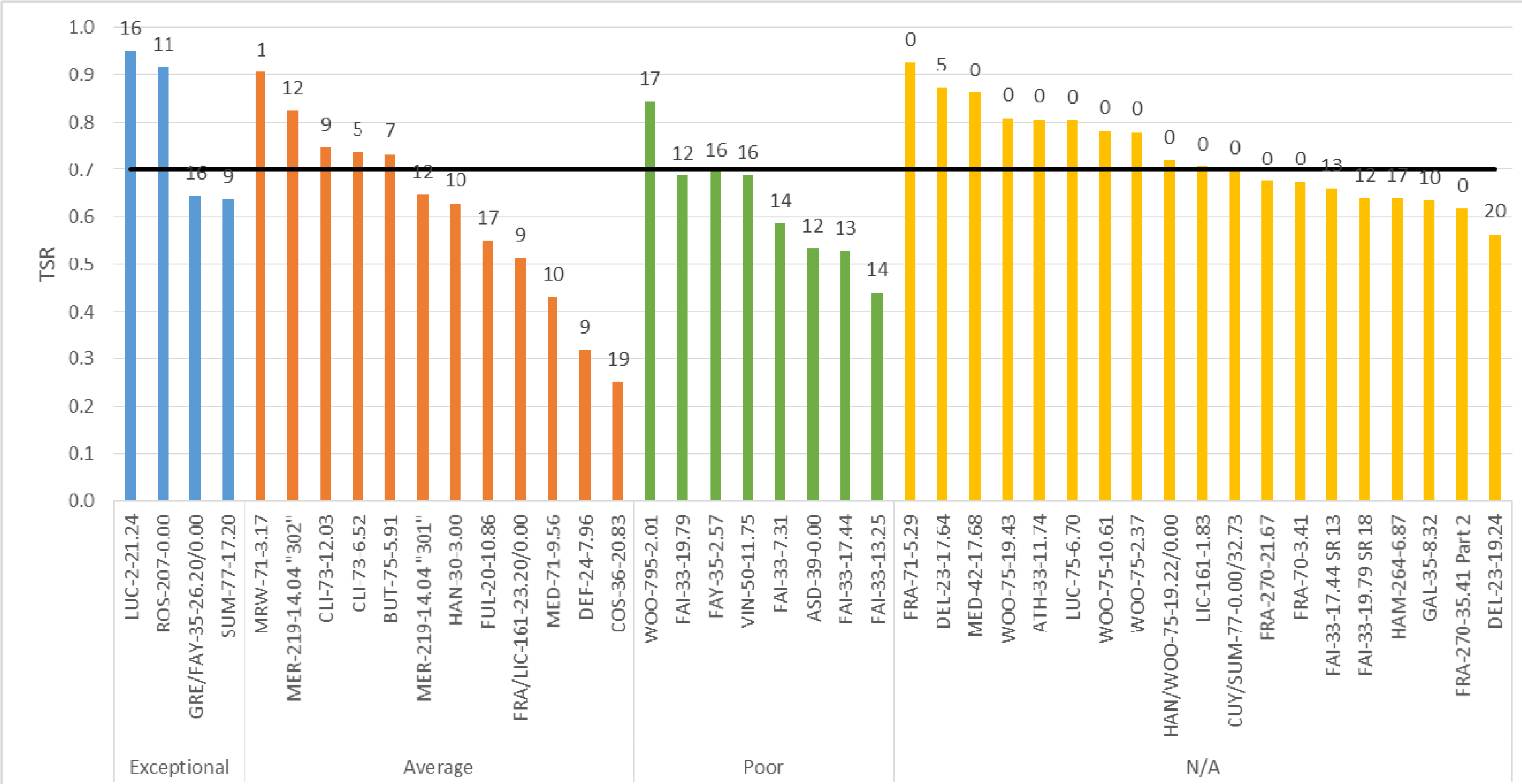


Figure 11-32. TSR by field performance category.

It is known that some aggregate types (e.g. gravel) are more prone to stripping, therefore the TSR values were summarized by the type of aggregate used in the AC base mixes. The average TSR, and number of projects meeting the criterion are shown in Table 11-23 for each aggregate type. Only 17% of the project sites with gravel AC base mixes met the 0.70 criterion, however there were fewer AC base mixes produced with gravel than limestone. On average, gravel AC base mixes were older than limestone mixes which as shown in Figure 11-31 may also have an impact.

Table 11–23. Summary of TSR values based on aggregate type used.

Aggregate Type	Number of Projects	Average TSR	Number of Projects TSR ≥ 0.7	Percent of Projects TSR ≥ 0.7	Average Pavement Age (yr)
Blend	9	0.63	3	33%	7.2
Gravel	6	0.66	1	17%	12.7
Limestone	22	0.70	12	55%	8.8
Unknown	6	0.70	3	50%	4.7

Lastly, TSR was plotted for each project site based on the type of recyled material used in the mix, as shown in Figure 11-33. Nearly 50% of the recyled mixes met the 0.70 criterion whereas only 25% of the virgin mixes had TSR values greater than or equal to 0.70. However, the number of recyled mixes far outweighs the number of virgin mixes. The use of RAP does not appear to have an effect on TSR for the AC base mixes tested here. However, the one RAS mix, sampled from DEF-24-7.96 had a very low TSR. While this mix had very high RAS content at 10% and low TSR, more data is needed to determine if there is an effect of RAS on the moisture suceptibility of Ohio's asphalt base mixes.

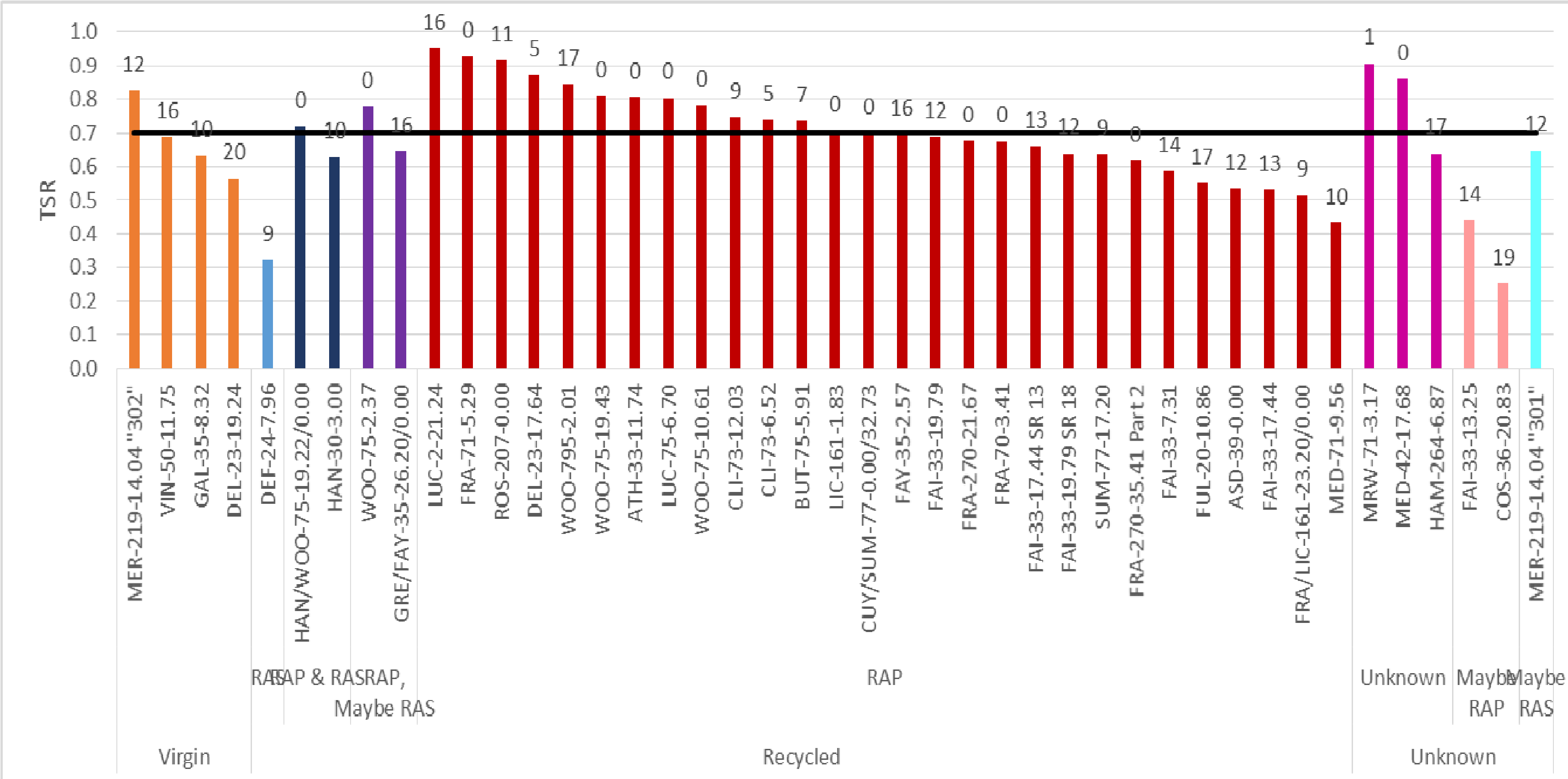


Figure 11-33. TSR values by use of recycled material.

Cantabro mass loss

As indicated by the literature review, Cantabro mass loss (ML) is a promising test for mix durability for dense graded asphalt mixes; low ML values should be indicative of durable mixes. Results for Cantabro testing are presented in Appendix L.

First, ML was plotted for each specimen against the measured specimen air voids, as shown in Figure 11-34. It was found that as specimen air voids increase, ML increases exponentially. An increase in mass loss with an increase in air voids is consistent with previous research conducted by Doyle and Howard [2010]. Although the R^2 associated with this trendline is only 0.42 it is still meaningful, as the data encompass mixes from across the state, mixes produced with and without recycled material, mixes containing different aggregate types, and mixes of varying age.

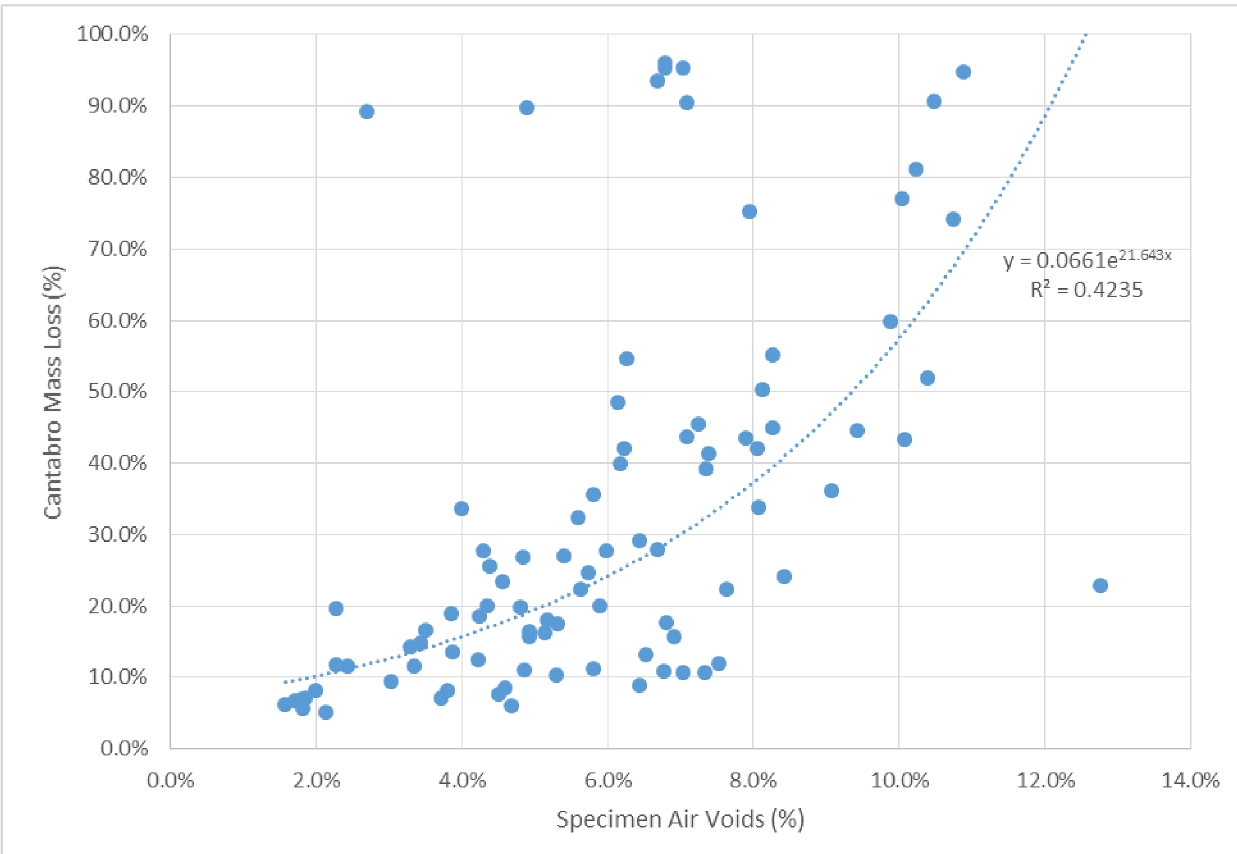


Figure 11-34. Cantabro mass loss vs specimen air voids.

ML results were plotted for each project site in order of increasing average ML in Figure 11-35. Results for the two specimens tested for each project site represent the minimum and maximum ML values and are shown as error bars in the plot. The TEX-245-F standard was followed which requires only two specimens, therefore the minimum and maximum ML values represent the two specimens tested from each project site. There is a wide range of mass loss values for the project sites tested, with some losing less than 10% mass, while others loss more than 90%. The two project sites for which the AC base mixes had more than 90% mass loss, COS-36-20.83 and DEF-24-7.96, also had some of the lowest TSR values.

While there are some exceptions, generally, as average ML increases, the spread in ML tends to increase as well. There appears to be a breakpoint at ML of 20%, where the spread (maximum minus minimum ML) is an average of 4.6% for projects sites with average ML less than 20% and an average of 25.7% for average ML greater than 20%. Of the 20 project sites with spread in ML greater than 10%, 19 had an average ML greater than 20%.

For some of the project sites with a wide spread in ML, segregation and/or poor compaction was observed in one of the samples. For example, Cantabro specimens for project site FAI-33-19.79 included Specimen 5A which had high air voids (8.3%) and showed signs of segregation, shown in Figure 11-36, and Specimen 13A, which had much lower air voids (3.5%) and appeared to have little to no segregation, shown in Figure 11-37. There was a notable difference in Cantabro ML for the two specimens. Specimen 5A had ML of 55.2%, more than three times the ML (16.6%) of Specimen 13A.

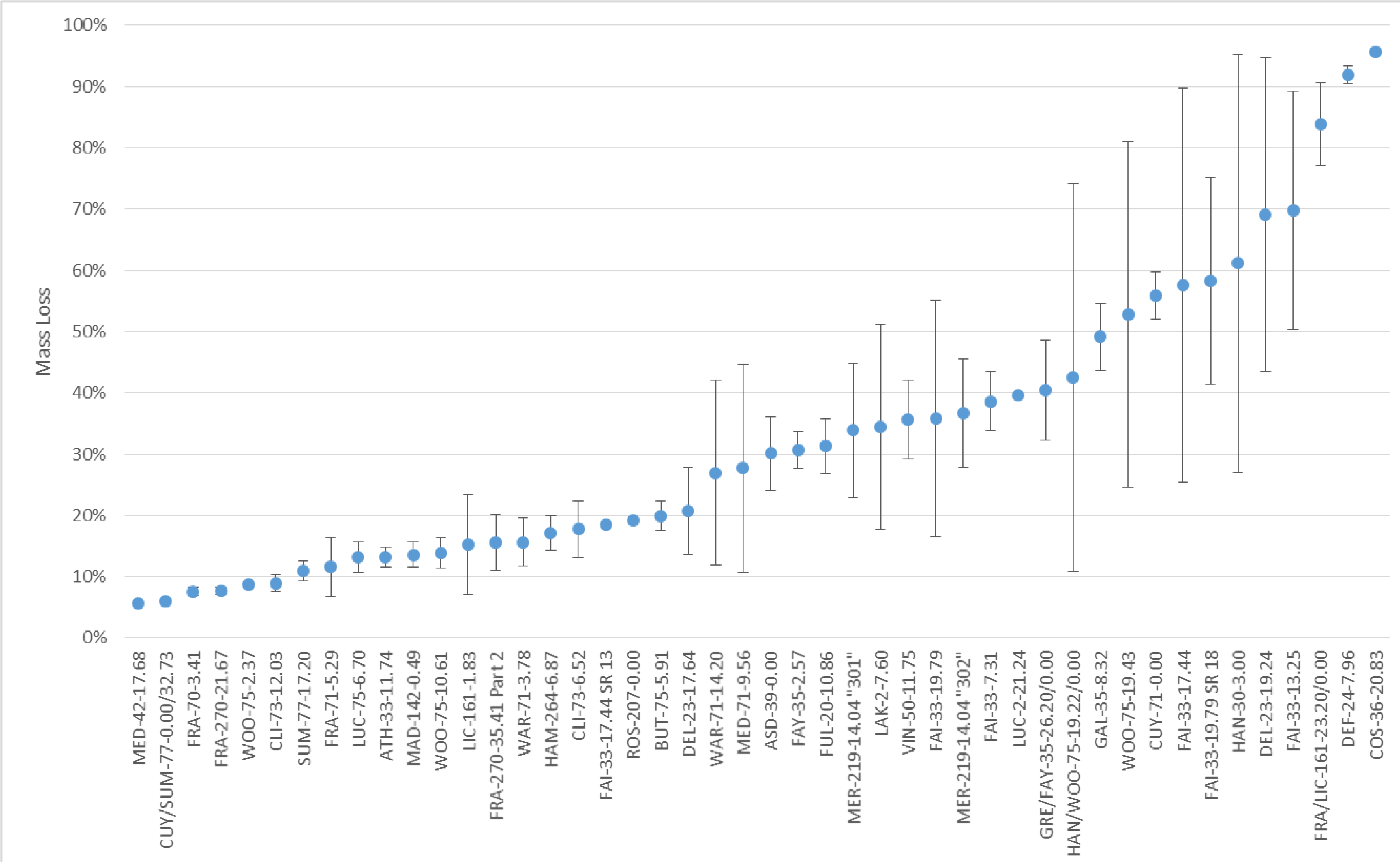


Figure 11-35. Cantabro mass loss by project site.

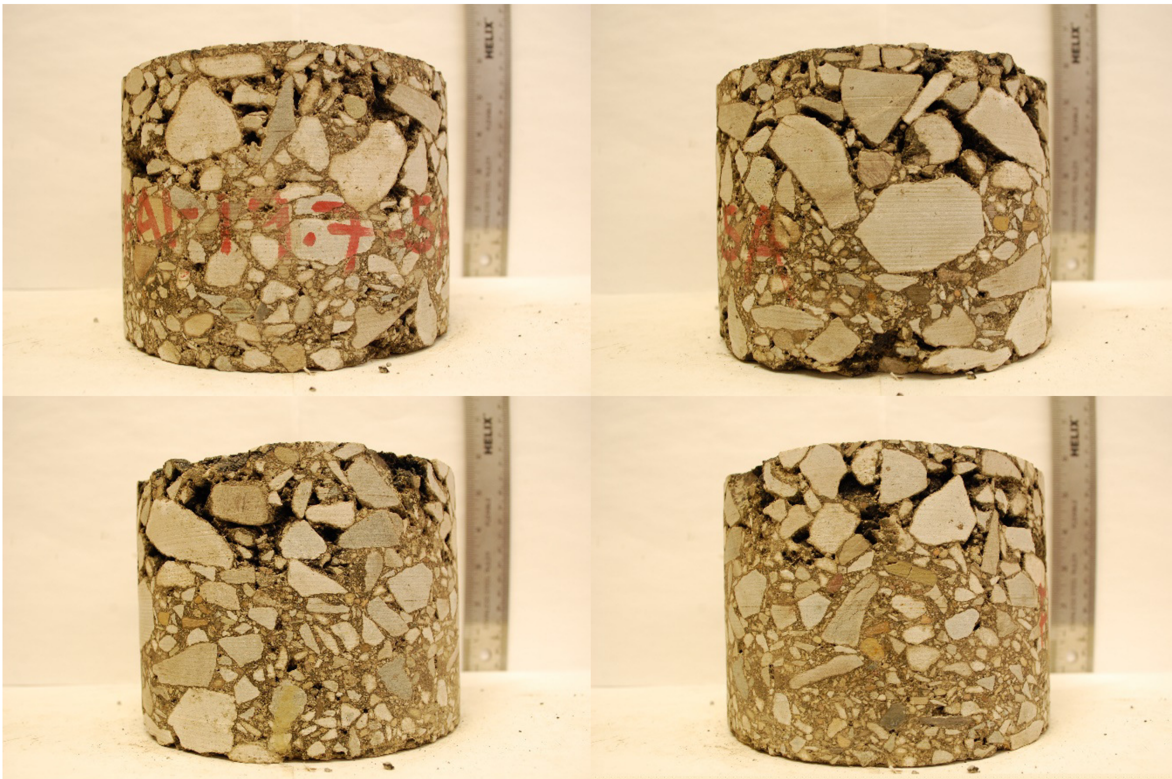


Figure 11-36. Cantabro sample with segregation and high ML, FAI-33-19.79, sample 5A.

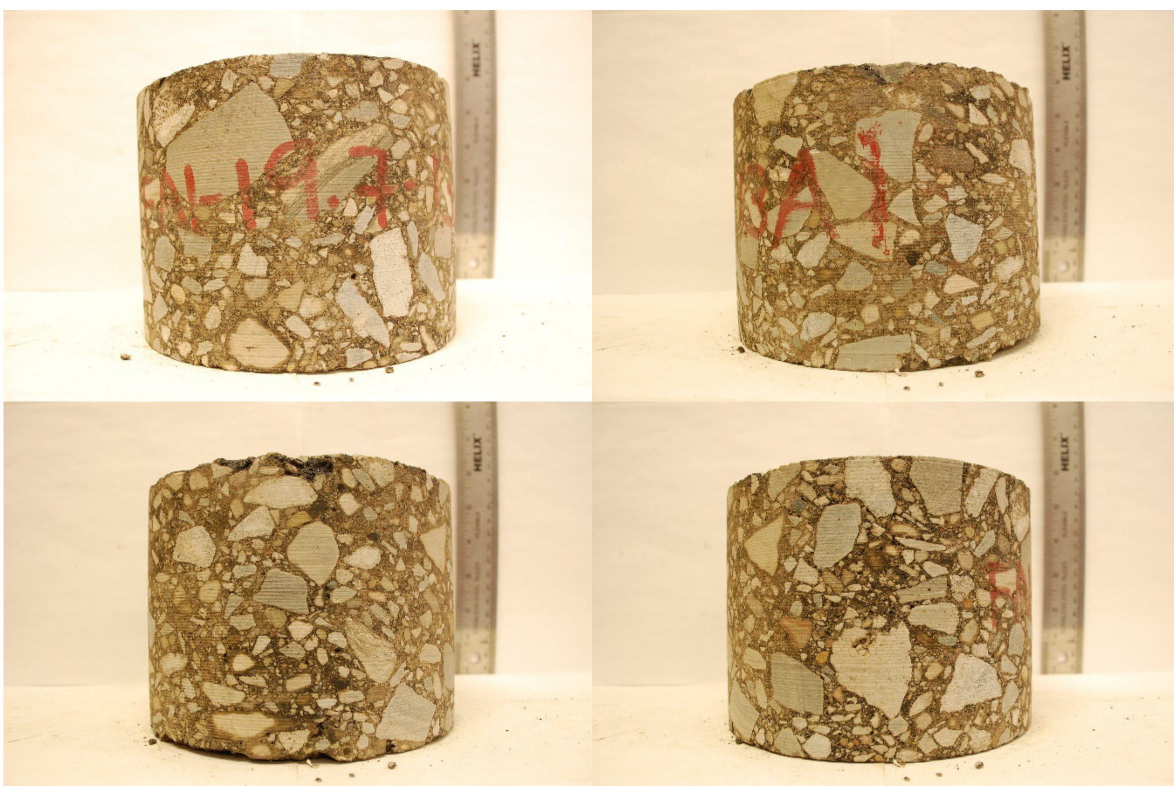


Figure 11-37. Cantabro sample with little to no segregation, and lower ML, FAI-33-19.79, sample 13A.

Based on the literature review, it is expected pavement age will have an effect on Cantabro mass loss, therefore average mass loss was plotted against pavement age, shown in Figure 11-38. The relationship shows an increase in Cantabro mass loss with age and for this investigation, was best fit with an exponential trend line. However, as evident by the R^2, the trend is not very strong. As shown in Figures 11-39 and 11-40, a stronger relationship between pavement age and average ML exists for ODOT item 302 base mixes. This could be related to the specification changes in design air voids for ODOT Item 302, as discussed under in-place air voids. Changes in the design air voids would likely have an effect on asphalt binder content which is well known to have an effect on durability.

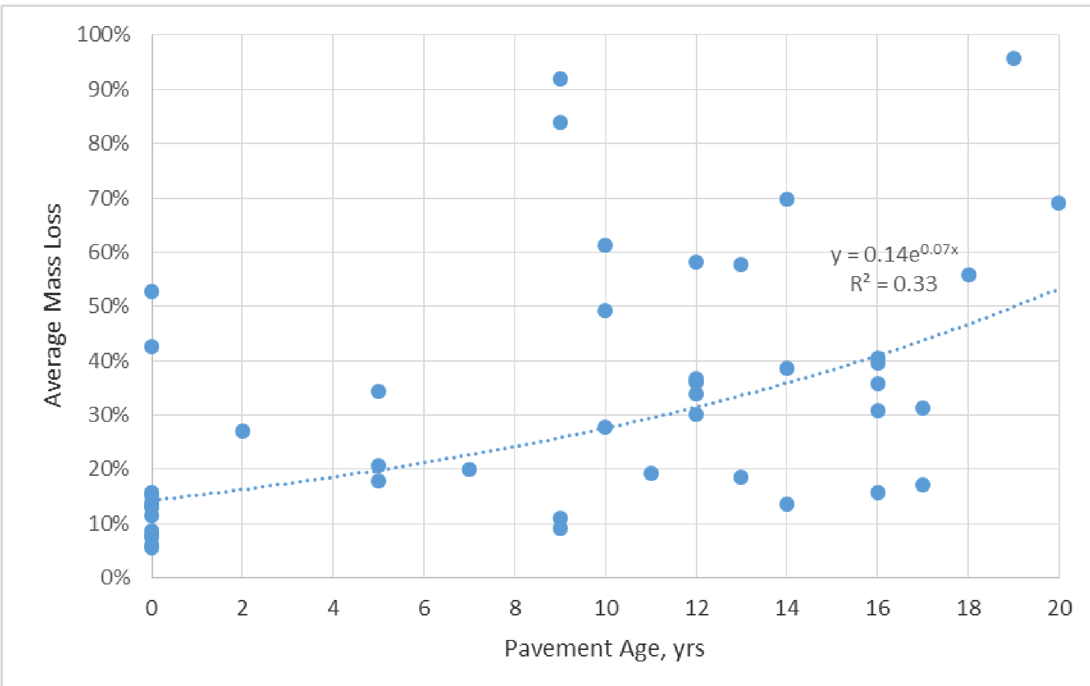


Figure 11-38. Average mass loss versus pavement age.

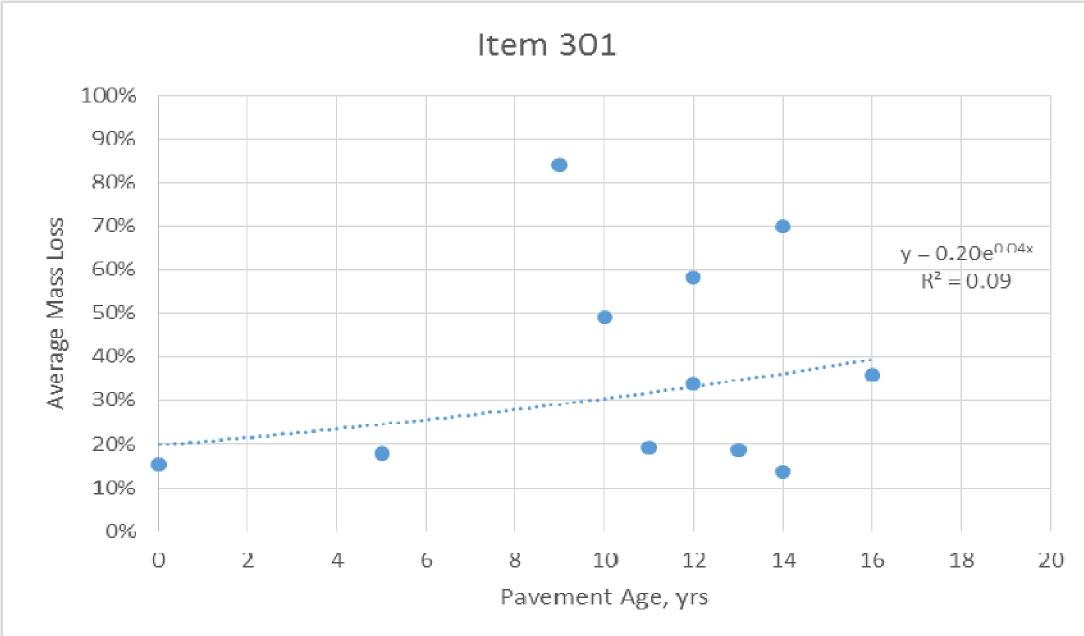


Figure 11-39. Average mass loss versus pavement age for ODOT Item 301.

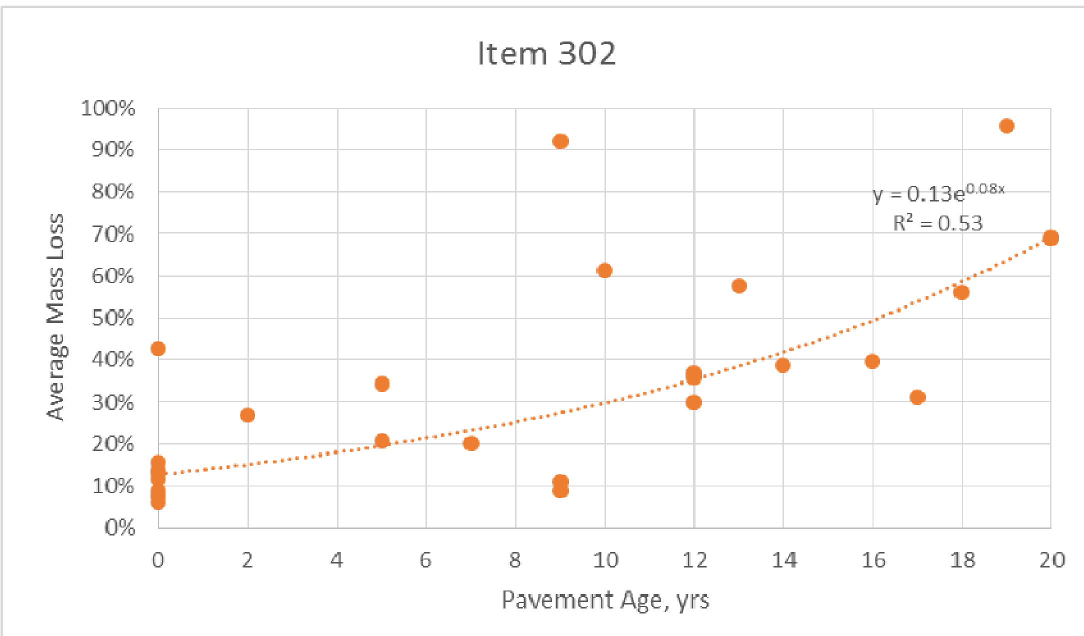


Figure 11-40. Average mass loss versus pavement age for ODOT Item 302.

The Cantabro mass loss was further broken down by age group, as shown in Table 11–24. As expected, based on the trend and previous literature, new AC bases had, on average, the lowest ML among all of the age groups and with the exception of bases aged 11 to 15 years old, the ML increases on average with each age group. Of the new pavements, 11 or 85% had ML less than 16%.

Table 11–24. Cantabro mass loss by age group.

Age (yr)	Number of Projects	Number of Samples	Average ML	Minimum ML	Maximum ML	Standard Deviation	Average Specimen Air Voids
0	13	26	16.4%	5.2%	81.1%	18.8%	4.7%
1 - 5	4	8	25.0%	11.9%	51.2%	14.6%	6.7%
6 - 10	8	16	44.2%	7.6%	95.3%	34.5%	6.5%
11 - 15	11	22	37.5%	11.5%	89.9%	23.0%	6.5%
16 - 20	10	20	43.1%	11.7%	96.0%	25.6%	6.1%

The effect of the use of recycled material was evaluated by plotting ML for each project site by use of recycled material and type of recycled material as shown in Figure 11-41. Each project site is also labeled with the pavement age since it was found above that age may influence ML. The virgin mixes had ML higher than expected, the four baes mixes tested were at least 10 years old. The second highest ML was for a 10% RAS mix placed at DEF-24-7.96. The use of RAP could not be determined for project site having the highest ML in the tested base mix. Those AC base mixes with RAS tended have higher ML, although there were only three mixes falling in the RAS or RAP and RAS categories. More data would be needed to determine if RAS truly has an impact on ML. There is a wide range of ML for AC base mixes produced with RAP, and it does not appear to be driven by age.

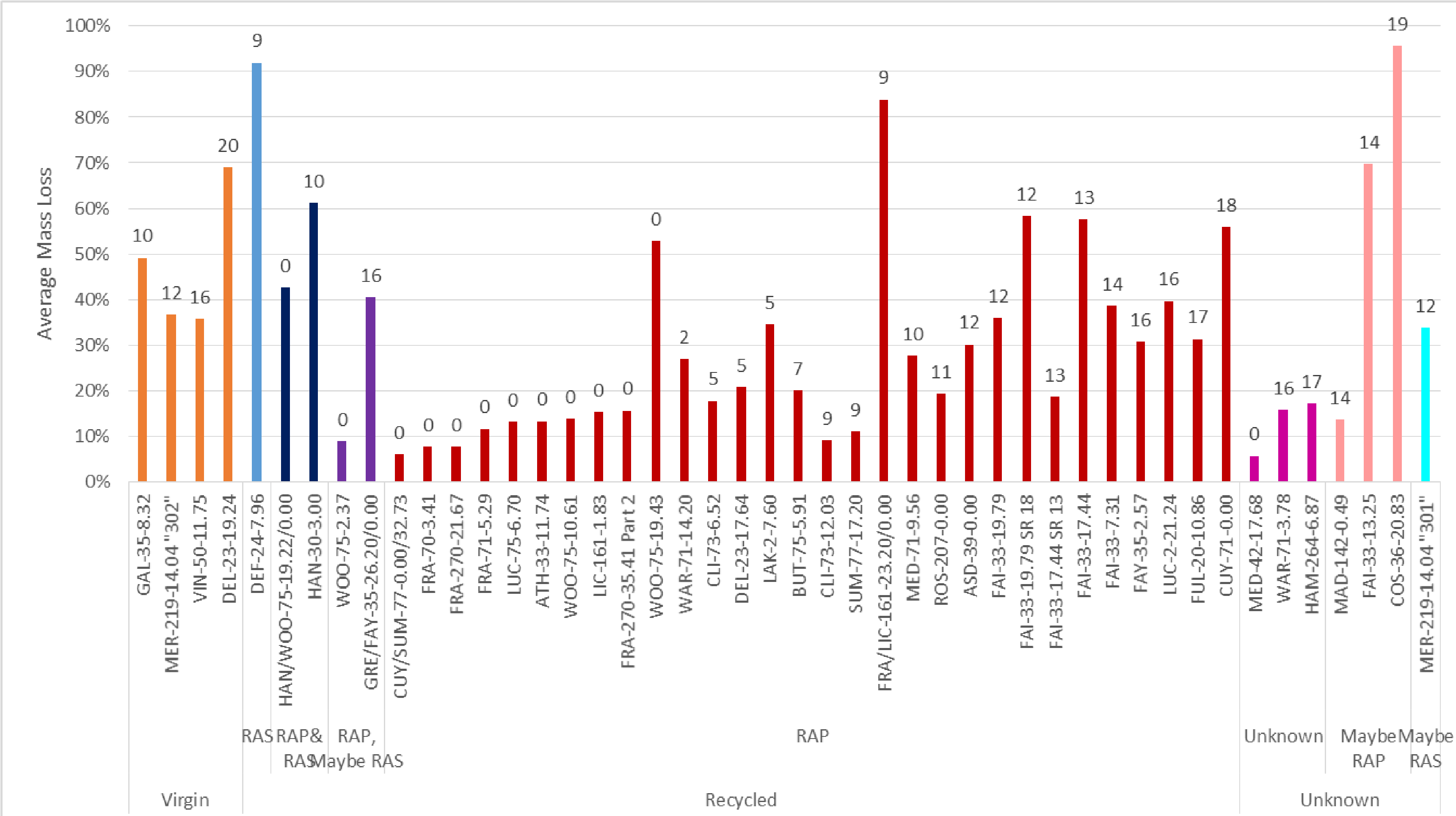


Figure 11-41. Cantabro mass loss by use of recycled material.

Lastly, Cantabro mass loss was evaluated by the aggregate type used in the AC base mix. Plotted in Figure 11-42 is the average ML for each project site based on the aggregate type used in the mix. The pavement age is also provided for each site. AC base mixes produced with a blend of gravel and limestone appeared to increase with pavement age, while this effect was not evident in the others. A large percentage of mixes were produced using limestone mixes, and there is a wide range of ML for mixes falling into this category. The highest ML was associated with a mix produced with blended aggregate, while the second highest was for a limestone mix.

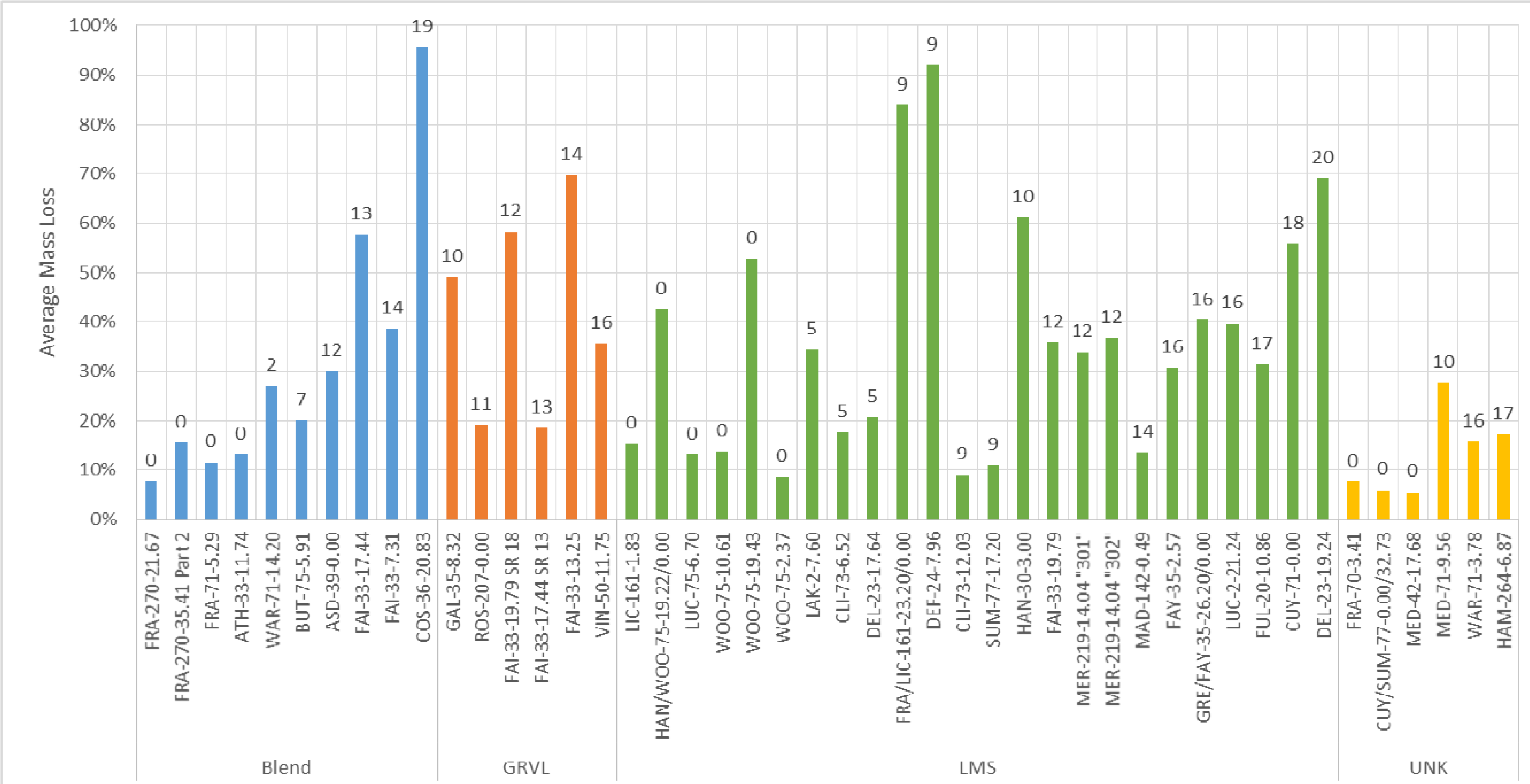


Figure 11-42. Average Cantabro mass loss by aggregate type used in AC base mix.

Comparisons with In-Place Air Voids

Once the results of the individual laboratory test were evaluated, the results were then compared among the different tests. First the relationship between average in-place air voids of the base mix for the project (as opposed to specimen air voids) and the test results were explored. Plotted against the average in-place air voids for the base mix for each site is the average Cantabro ML in Figure 11-43, average FI in Figure 11-44, and TSR value in Figure 11-45. As was expected based on the relationship found between air voids and ML measured for each specimen, there was a moderate relationship found between average in-place air voids and average ML for each project. A downward trend was found for both TSR and average FI such that FI and TSR tend to decrease with an increase in average in-place air voids. However, in both cases the relationship was weak.

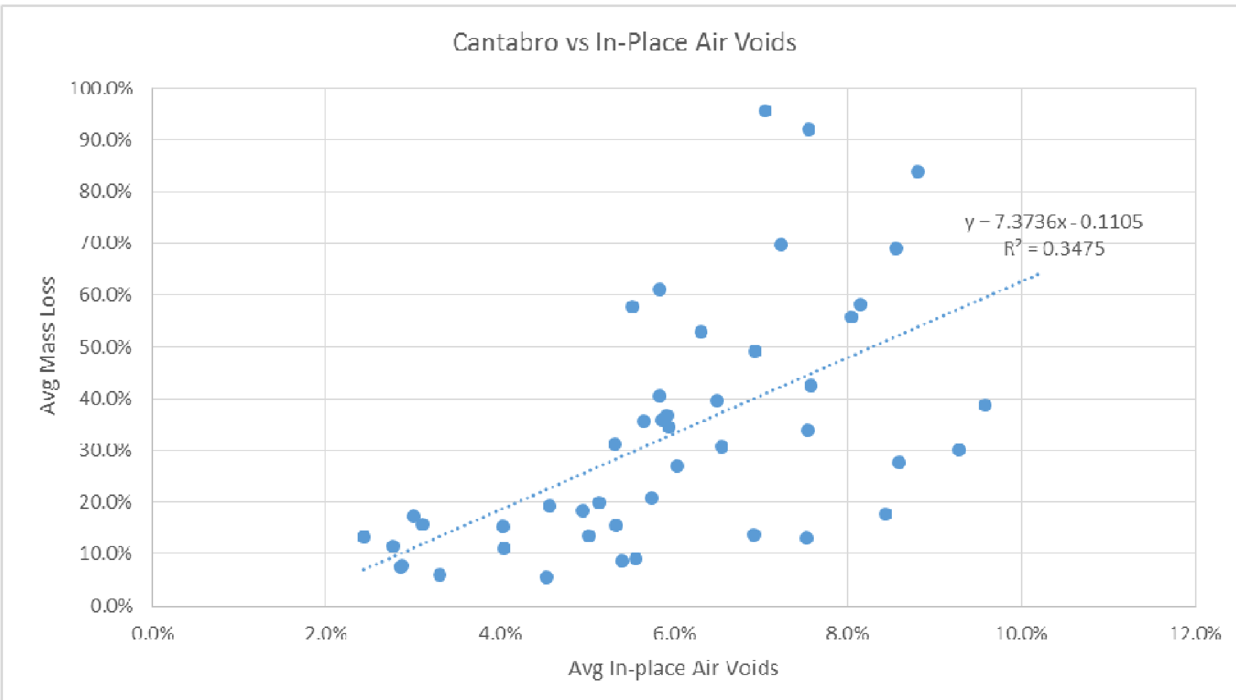


Figure 11-43. Average mass loss vs average in-place air voids.

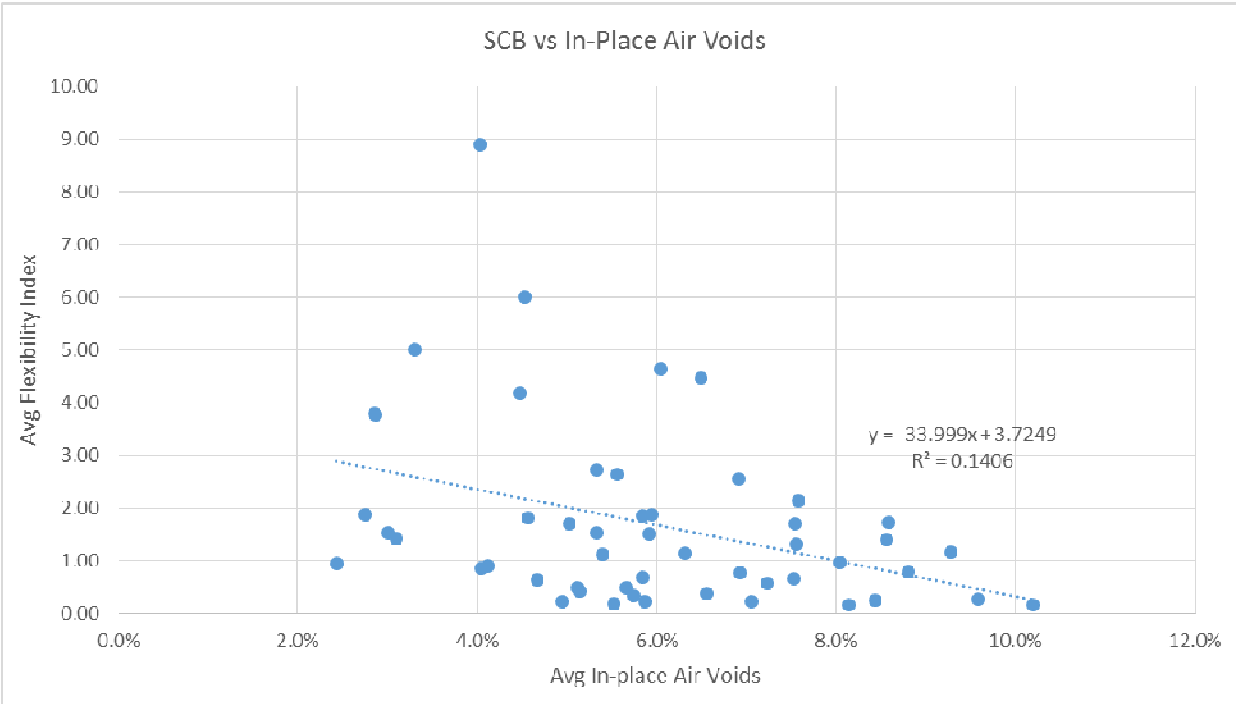


Figure 11-44. Average flexibility index vs average in-place air voids.

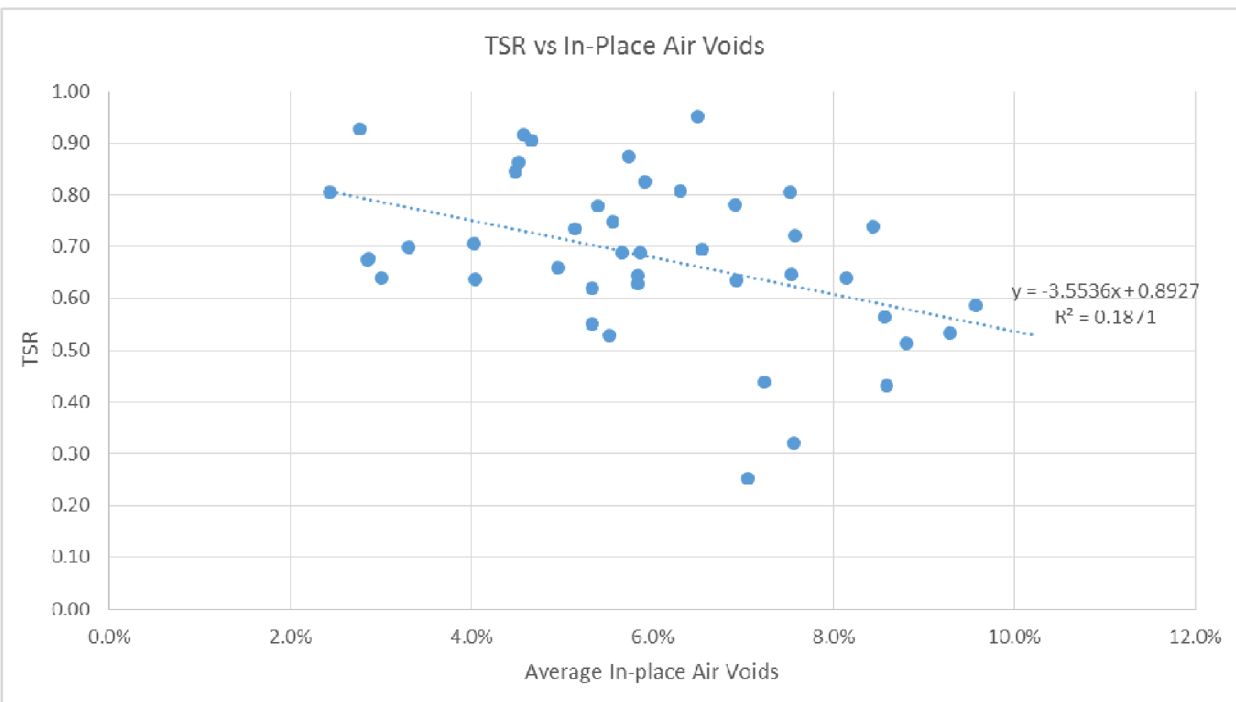


Figure 11-45. TSR vs average in-place air voids.

Comparisons among Laboratory Tests

Next, test results were plotted against one another to determine if there was any relationship among them. Average FL was plotted against average ML as shown in Figure 11-46. Although the trend shows a decrease in average FI with an increase in average ML, the relationship is weak. However, there is a separation in the data such that where the average ML was 40% or more, the average FI was less than 2.1.

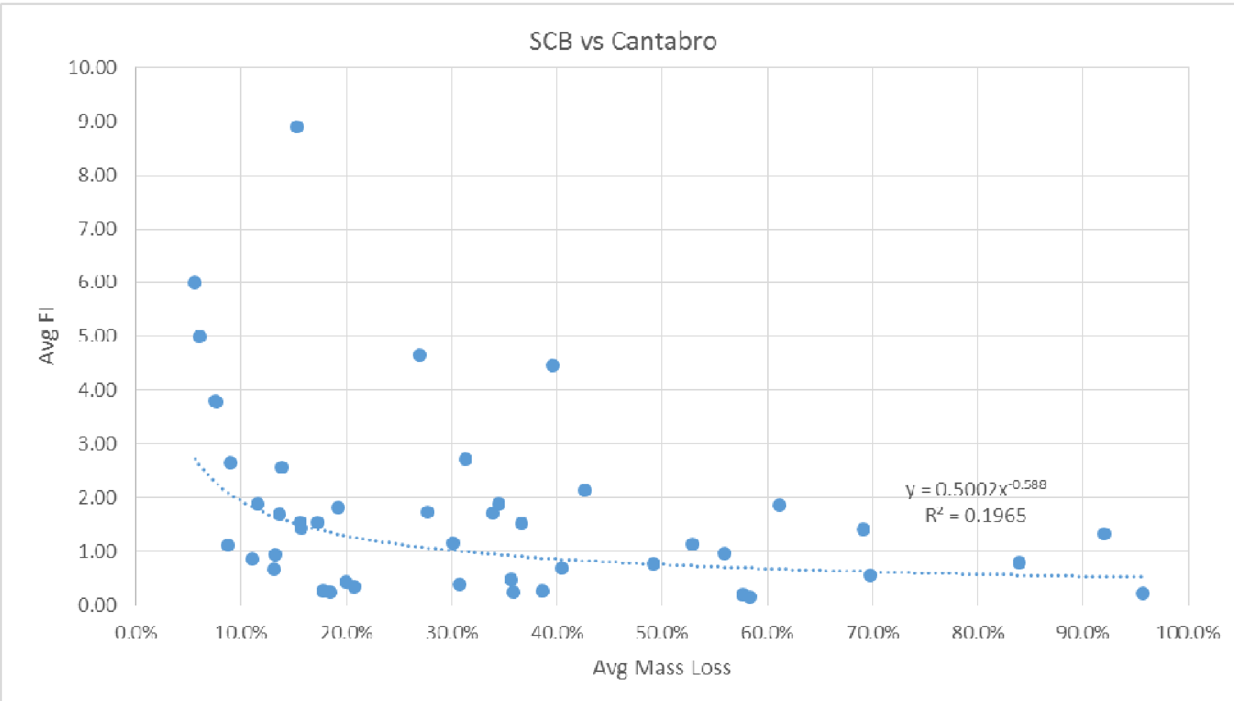


Figure 11-46. Average FI vs average ML.

In comparing TSR with average FI in Figure 11-47, higher FI was associated with higher TSR values, however the relationship is very weak. When average dry tensile strength is plotted against average FI, as shown in Figure 11-48, the relationship improves. It appears from the plot that lower FI values are associated with higher dry tensile strength up to an FI of approximately 2 or dry tensile strength of 150 psi (1,034 kPa) after which the curve flattens out.

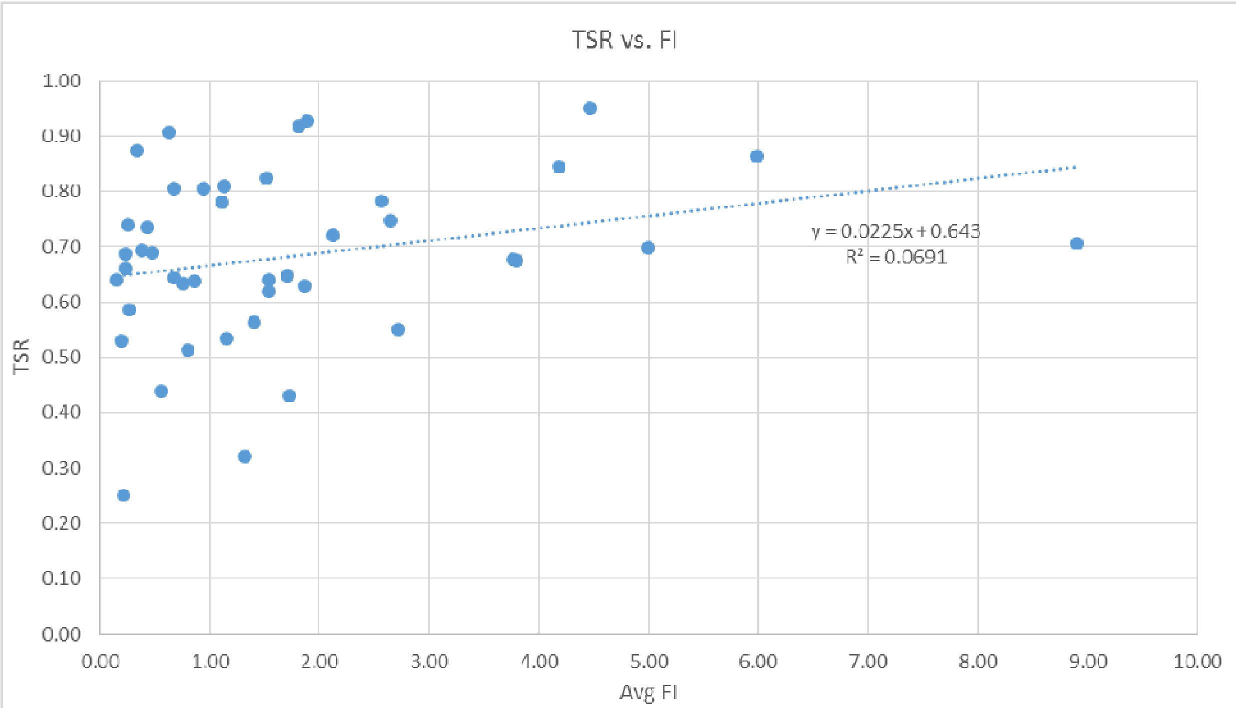


Figure 11-47. TSR vs average FI.

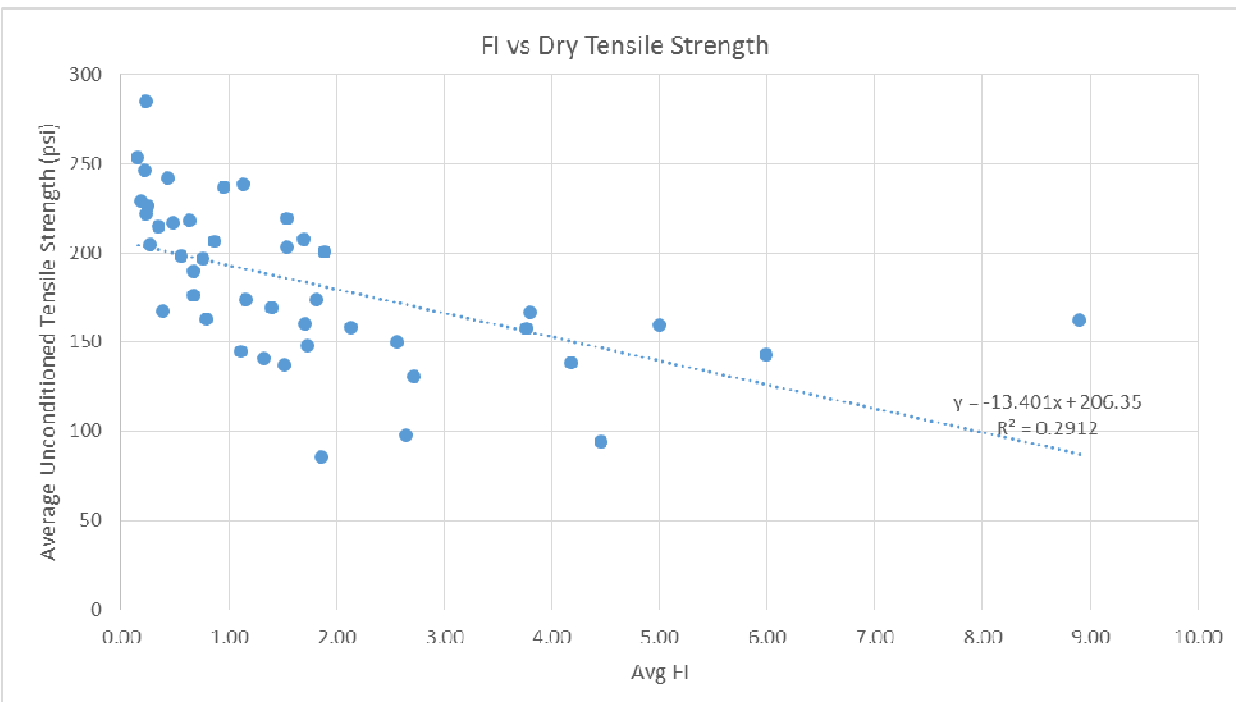


Figure 11-48. Dry tensile strength vs average FI (1 psi = 6.89 kPa).

Lastly, TSR values were plotted against average ML in Figure 11-49. This plot shows a moderately strong relationship such that an increase in ML is associated with a decrease in TSR. This is expected as a durable mix should also be resistance to moisture damage. There appears to be a separation in the data such that ML greater than 21% tends to have TSR values less than the criterion of 0.70.

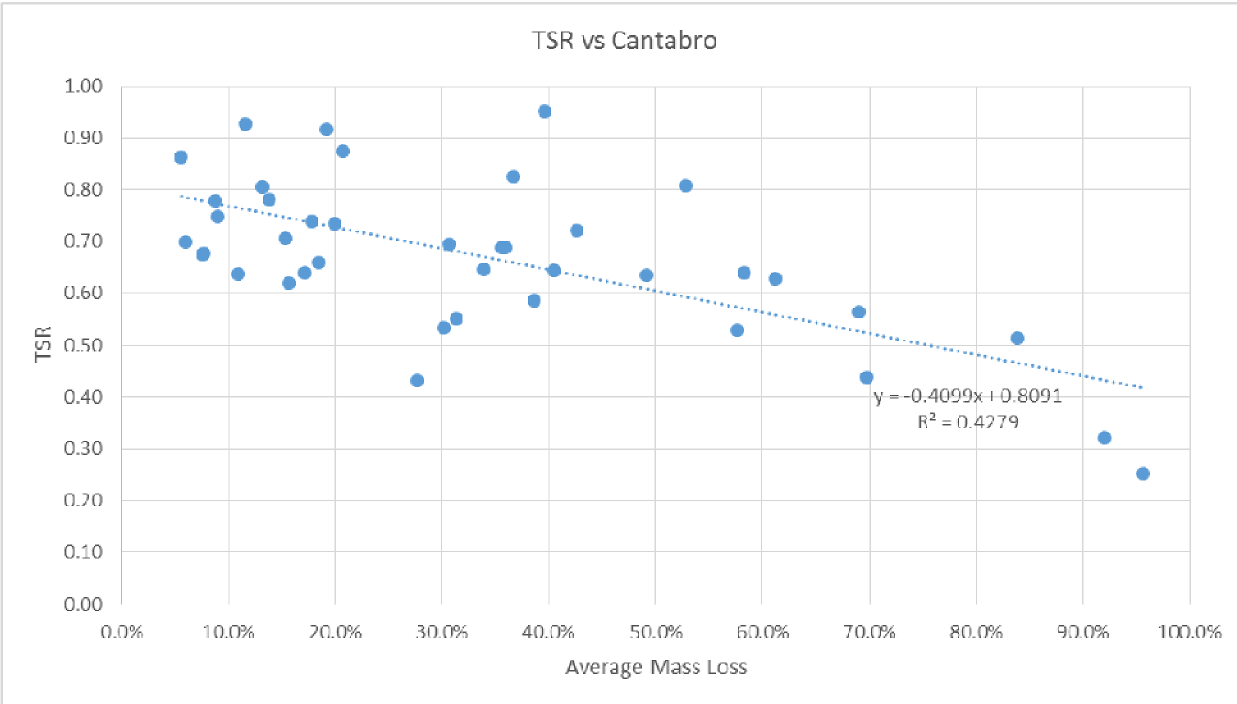


Figure 11-49. TSR vs average ML.

Appendix J: SCB Test Results

Table 11–25. Summary of IL-SCB results.

Project ID	Number of Specimens	Average Air voids	Average Fracture Energy (J/m^2)	Average Fracture Energy (BTU/ft^2)	Average FI	Minimum FI	Maximum FI	StdDev of FI	CoV of FI
ASD-39-0.00	6	10.18%	1311.9	0.1154	1.16	0.43	2.61	0.84	72.9%
ATH-33-11.74	6	2.82%	1583.4	0.1393	0.95	0.25	2.19	0.85	89.3%
BUT-75-5.91	6	5.61%	846.0	0.0744	0.43	0.15	1.57	0.56	130.5%
CLA-70-13.98	8	4.12%	985.6	0.0867	0.90	0.15	2.84	0.96	106.2%
CLI-73-12.03	6	8.25%	824.7	0.0726	2.65	0.86	4.95	1.78	67.3%
CLI-73-6.52	6	7.81%	692.2	0.0609	0.25	0.17	0.44	0.10	38.6%
COS-36-20.83	6	7.13%	795.0	0.0700	0.22	0.13	0.54	0.16	73.5%
CUY/SUM-77-0.00/32.73	6	3.96%	1892.2	0.1665	5.00	1.97	7.13	1.99	39.8%
CUY-71-0.00	6	7.58%	828.8	0.0729	0.97	0.16	3.84	1.42	146.7%
DEF-24-7.96	6	8.09%	486.4	0.0428	1.32	0.49	3.52	1.13	85.8%
DEL-23-17.64	6	4.63%	912.8	0.0803	0.34	0.08	0.97	0.32	94.4%
DEL-23-19.24	6	8.03%	835.3	0.0735	1.41	0.44	2.35	0.75	53.2%
FAI-33-13.25	6	6.41%	700.9	0.0617	0.56	0.24	0.98	0.24	42.3%
FAI-33-17.44	6	4.94%	679.3	0.0598	0.19	0.13	0.29	0.06	30.0%
FAI-33-17.44 SR 13	6	3.70%	863.6	0.0760	0.23	0.19	0.29	0.03	14.1%
FAI-33-19.79	10	6.67%	731.2	0.0643	0.23	0.10	0.57	0.16	68.8%
FAI-33-19.79 SR 18	6	9.91%	507.8	0.0447	0.15	0.12	0.25	0.05	31.5%
FAI-33-7.31	6	9.94%	509.8	0.0449	0.27	0.10	0.59	0.18	69.0%
FAY-35-2.57	6	9.02%	762.4	0.0671	0.39	0.12	0.73	0.25	65.7%
FRA/LIC-161-23.20/0.00	8	10.20%	639.9	0.0563	0.80	0.10	2.44	0.82	103.4%
FRA-270-21.67	6	2.02%	2082.7	0.1833	3.77	2.22	5.18	1.43	37.9%
FRA-270-35.41 Part 2	6	4.44%	1305.0	0.1148	1.54	0.72	2.47	0.72	46.6%
FRA-70-3.41	8	2.63%	1948.0	0.1714	3.80	2.23	5.30	1.19	31.4%
FRA-71-5.29	11	3.00%	1403.8	0.1235	1.89	0.44	3.63	0.98	51.9%
FUL-20-10.86	6.00	5.71%	862.8	0.0759	2.72	1.00	5.12	1.59	58.5%

Project ID	Number of Specimens	Average Air voids	Average Fracture Energy (J/m^2)	Average Fracture Energy (BTU/ft^2)	Average FI	Minimum FI	Maximum FI	StdDev of FI	CoV of FI
GAL-35-8.32	10	7.51%	1067.4	0.0939	0.76	0.19	2.14	0.72	95.1%
GRE/FAY-35-26.20/0.00	6	6.29%	849.5	0.0748	0.68	0.16	1.42	0.49	73.2%
HAM-264-6.87	8	4.00%	1472.3	0.1296	1.54	0.30	3.27	1.10	71.8%
HAN/WOO-75-19.22/0.00	6	7.72%	1308.8	0.1152	2.13	1.17	3.31	0.99	46.5%
HAN-30-3.00	6	5.80%	757.9	0.0667	1.86	0.70	3.60	1.19	63.9%
LAK-2-7.60	14	5.13%	1472.2	0.1296	1.89	0.27	9.60	2.53	134.1%
LIC-161-1.83	8	4.57%	2240.6	0.1972	8.90	4.95	15.76	3.20	35.9%
LUC-2-21.24	8	6.09%	1182.0	0.1040	4.46	2.45	6.50	1.46	32.7%
LUC-75-6.70	6	6.92%	1171.4	0.1031	0.67	0.28	1.03	0.30	44.5%
MAD-142-0.49	8	5.06%	1216.1	0.1070	1.69	0.50	3.29	0.94	55.6%
MED-42-17.68	6	4.52%	2098.9	0.1847	6.00	4.05	7.34	1.38	23.0%
MED-71-9.56	6	9.74%	878.6	0.0773	1.73	0.90	3.63	1.00	58.0%
MER-219-14.04 "301"	6	7.02%	818.4	0.0720	1.71	0.68	4.40	1.41	82.2%
MER-219-14.04 "302"	10	8.45%	505.2	0.0445	1.52	0.24	2.69	0.82	54.3%
MRW-71-3.17	8	5.26%	831.0	0.0731	0.64	0.30	1.69	0.50	78.0%
ROS-207-0.00	8	4.64%	1331.2	0.1171	1.81	0.21	6.20	2.08	114.7%
SUM-77-17.20	6	3.92%	1075.0	0.0946	0.86	0.23	1.58	0.51	58.7%
VIN-50-11.75	6	5.69%	594.4	0.0523	0.48	0.21	0.88	0.26	55.0%
WAR-71-14.20	6	6.24%	1679.7	0.1478	4.65	2.31	8.16	2.06	44.4%
WAR-71-3.78	6	3.72%	1178.6	0.1037	1.43	0.25	3.10	1.10	77.1%
WAY-30-11.86 "301"	4	9.76%	419.2	0.0369	0.16	0.12	0.20	0.03	21.6%
WAY-30-11.86 "302"	8	5.95%	865.1	0.0761	0.49	0.26	1.06	0.29	58.4%
WOO-75-10.61	6	7.73%	991.3	0.0872	2.56	0.82	4.71	1.65	64.4%
WOO-75-19.43	6	5.82%	1328.4	0.1169	1.13	0.38	1.90	0.57	50.2%
WOO-75-2.37	6	6.04%	1140.4	0.1004	1.11	0.44	1.96	0.55	49.5%
WOO-795-2.01	6	5.59%	871.8	0.0767	4.18	2.54	8.50	2.20	52.6%

Table 11–26. SCB-IL complete results.

Project ID	Specimen ID	Specimen Air voids	FI	Fracture Energy (J/m^2)	(BTU/ft^2)
ASD-39-0.00	16A1-1	9.87%	1.66	849.37	0.0747
ASD-39-0.00	16A1-2	8.98%	2.61	3298.69	0.2903
ASD-39-0.00	3A1-1	10.64%	0.45	891.88	0.0785
ASD-39-0.00	3A1-2	11.01%	0.43	784.08	0.0690
ASD-39-0.00	6A1-1	10.68%	0.97	1005.09	0.0884
ASD-39-0.00	6A1-2	9.87%	0.81	1042.2	0.0917
ATH-33-11.74	13A1-1	3.23%	2.19	2196.27	0.1933
ATH-33-11.74	13A1-2	2.30%	1.82	1869.98	0.1646
ATH-33-11.74	3A1-1	2.54%	0.77	1520.67	0.1338
ATH-33-11.74	3A1-2	3.43%	0.30	1298.41	0.1143
ATH-33-11.74	6A1-1	3.07%	0.25	1090.31	0.0959
ATH-33-11.74	6A1-2	2.34%	0.36	1524.97	0.1342
BUT-75-5.91	12A1-1	6.35%	1.57	1138.09	0.1002
BUT-75-5.91	12A1-2	6.63%	0.24	907.94	0.0799
BUT-75-5.91	4A1-1	5.51%	0.15	499.57	0.0440
BUT-75-5.91	4A1-2	4.59%	0.15	537.22	0.0473
BUT-75-5.91	9A1-1	5.47%	0.15	688.89	0.0606
BUT-75-5.91	9A1-2	5.11%	0.33	1304.06	0.1148
CLA-70-13.98	2A1-1	4.98%	0.19	684.41	0.0602
CLA-70-13.98	2A1-2	5.02%	0.15	642.28	0.0565
CLA-70-13.98	2A2-1	4.59%	0.27	762.67	0.0671
CLA-70-13.98	2A2-2	3.80%	0.83	854.15	0.0752
CLA-70-13.98	5A1-1	4.47%	0.75	1121.41	0.0987
CLA-70-13.98	5A1-2	3.80%	0.35	906.77	0.0798
CLA-70-13.98	9A2-1	2.97%	1.82	1533.42	0.1349
CLA-70-13.98	9A2-2	3.36%	2.84	1379.36	0.1214
CLI-73-6.52	1A1-1	9.56%	0.44	1009.59	0.0888
CLI-73-6.52	1A1-2	9.04%	0.23	593.61	0.0522
CLI-73-6.52	5A1-1	6.04%	0.27	792.71	0.0698
CLI-73-6.52	5A1-2	7.24%	0.20	698.85	0.0615
CLI-73-6.52	8A1-1	7.80%	0.21	639.28	0.0563
CLI-73-6.52	8A1-2	7.20%	0.17	419.27	0.0369
CLI-73-12.03	10A1-1	7.97%	4.95	1063.98	0.0936
CLI-73-12.03	10A1-2	8.84%	1.14	551.01	0.0485
CLI-73-12.03	13A1-1	7.85%	1.19	778.65	0.0685
CLI-73-12.03	13A1-2	8.37%	0.86	370.63	0.0326
CLI-73-12.03	3A1-1	8.49%	3.91	1040.4	0.0916
CLI-73-12.03	3A1-2	8.01%	3.83	1143.69	0.1006
COS-36-20.83	15A1-1	9.66%	0.15	360.91	0.0318

Project ID	Specimen ID	Specimen Air voids	FI	Fracture Energy (J/m^2)	(BTU/ft^2)
COS-36-20.83	15A1-2	8.65%	0.17	685.12	0.0603
COS-36-20.83	4A1-1	6.51%	0.54	2316.12	0.2038
COS-36-20.83	4A1-2	5.82%	0.13	404.6	0.0356
COS-36-20.83	7A1-1	5.90%	0.15	515.38	0.0454
COS-36-20.83	7A1-2	6.27%	0.16	487.68	0.0429
CUY-71-0.00	11A1-1	6.57%	0.47	566.58	0.0499
CUY-71-0.00	11A1-2	7.01%	0.53	489.05	0.0430
CUY-71-0.00	1A1-1	8.80%	0.29	642.02	0.0565
CUY-71-0.00	1A1-2	7.32%	3.84	2233.65	0.1966
CUY-71-0.00	5A1-1	7.21%	0.50	608.49	0.0535
CUY-71-0.00	5A1-2	8.56%	0.16	432.96	0.0381
CUY/SUM-77-0.00/32.73	13A1-1	3.09%	7.11	2295.62	0.2020
CUY/SUM-77-0.00/32.73	13A1-2	3.68%	3.96	1846.44	0.1625
CUY/SUM-77-0.00/32.73	1A1-1	3.21%	7.13	2123.68	0.1869
CUY/SUM-77-0.00/32.73	1A1-2	3.52%	5.43	1998.1	0.1758
CUY/SUM-77-0.00/32.73	7A1-1	4.75%	4.41	1903	0.1675
CUY/SUM-77-0.00/32.73	7A1-2	5.50%	1.97	1186.4	0.1044
DEF-24-7.96	11A1-1	7.32%	0.85	390.66	0.0344
DEF-24-7.96	11A1-2	6.84%	0.73	479.18	0.0422
DEF-24-7.96	14A1-1	7.80%	3.52	1147.27	0.1010
DEF-24-7.96	14A1-2	7.88%	0.49	487.72	0.0429
DEF-24-7.96	1A1-1	8.72%	1.53	178.77	0.0157
DEF-24-7.96	1A1-2	9.96%	0.80	235.07	0.0207
DEL-23-17.64	14A1-1	4.80%	0.18	785.02	0.0691
DEL-23-17.64	14A1-2	4.76%	0.24	815.16	0.0717
DEL-23-17.64	6A1-1	3.48%	0.97	1647.68	0.1450
DEL-23-17.64	6A1-2	3.68%	0.35	1140.44	0.1004
DEL-23-17.64	9A1-1	5.76%	0.22	757.31	0.0666
DEL-23-17.64	9A1-2	5.32%	0.08	331.03	0.0291
DEL-23-19.24	12A1-1	6.43%	0.87	968.48	0.0852
DEL-23-19.24	12A1-2	5.96%	0.44	779.32	0.0686
DEL-23-19.24	5A1-1	8.34%	2.35	694.52	0.0611
DEL-23-19.24	5A1-2	9.21%	2.21	910.92	0.0802
DEL-23-19.24	8A1-1	9.61%	1.27	939.57	0.0827
DEL-23-19.24	8A1-2	8.62%	1.29	718.78	0.0633
FAI-33-7.31	13A1-1	8.66%	0.10	381.48	0.0336
FAI-33-7.31	13A1-2	8.30%	0.12	412.52	0.0363
FAI-33-7.31	2A1-1	9.57%	0.59	454.97	0.0400
FAI-33-7.31	2A1-2	8.66%	0.16	472.73	0.0416

Project ID	Specimen ID	Specimen Air voids	FI	Fracture Energy (J/m^2)	(BTU/ft^2)
FAI-33-7.31	8A1-1	12.21%	0.30	284.52	0.0250
FAI-33-7.31	8A1-2	12.25%	0.33	1052.49	0.0926
FAI-33-13.25	10A1-1	6.57%	0.54	985.69	0.0867
FAI-33-13.25	10A1-2	8.16%	0.50	223.32	0.0197
FAI-33-13.25	14A1-1	5.81%	0.59	886.11	0.0780
FAI-33-13.25	14A1-2	6.76%	0.24	342.14	0.0301
FAI-33-13.25	1A1-1	5.49%	0.53	851.2	0.0749
FAI-33-13.25	1A1-2	5.69%	0.98	917.01	0.0807
FAI-33-17.44	11A1-1	5.54%	0.15	713.94	0.0628
FAI-33-17.44	11A1-2	5.66%	0.13	478.16	0.0421
FAI-33-17.44	2A1-1	4.98%	0.21	891.08	0.0784
FAI-33-17.44	2A1-2	4.70%	0.17	663.04	0.0583
FAI-33-17.44	7A1-1	4.26%	0.20	679.97	0.0598
FAI-33-17.44	7A1-2	4.54%	0.29	649.51	0.0572
FAI-33-19.79	10A1-1	4.11%	0.41	744.14	0.0655
FAI-33-19.79	10A1-2	3.11%	0.19	570.93	0.0502
FAI-33-19.79	3A1-1	7.98%	0.10	429.75	0.0378
FAI-33-19.79	3A1-2	8.78%	0.10	272.39	0.0240
FAI-33-19.79	3A2-1	7.90%	0.14	648.33	0.0571
FAI-33-19.79	3A2-2	8.54%	0.35	361.44	0.0318
FAI-33-19.79	6A1-1	7.26%	0.13	505.05	0.0444
FAI-33-19.79	6A1-2	6.82%	0.15	669.38	0.0589
FAI-33-19.79	6A2-1	6.14%	0.17	627.96	0.0553
FAI-33-19.79	6A2-2	6.10%	0.57	2482.53	0.2185
FAI-33-17.44 SR 13	12A1-1	4.10%	0.22	920.3	0.0810
FAI-33-17.44 SR 13	12A1-2	1.16%	0.23	790.79	0.0696
FAI-33-17.44 SR 13	1A1-1	2.81%	0.22	925.31	0.0814
FAI-33-17.44 SR 13	1A1-2	2.13%	0.19	650.18	0.0572
FAI-33-17.44 SR 13	7A1-1	6.02%	0.29	951.21	0.0837
FAI-33-17.44 SR 13	7A1-2	5.98%	0.25	943.63	0.0830
FAI-33-19.79 SR 18	11A1-1	9.86%	0.12	404.62	0.0356
FAI-33-19.79 SR 18	11A1-2	9.57%	0.14	438.39	0.0386
FAI-33-19.79 SR 18	2A1-1	8.44%	0.16	463.42	0.0408
FAI-33-19.79 SR 18	2A1-2	7.75%	0.14	535.68	0.0471
FAI-33-19.79 SR 18	7A1-1	12.21%	0.25	773.83	0.0681
FAI-33-19.79 SR 18	7A1-2	11.60%	0.12	431.11	0.0379
FAY-35-2.57	11A1-1	8.53%	0.25	703.37	0.0619
FAY-35-2.57	11A1-2	8.22%	0.61	1356.81	0.1194
FAY-35-2.57	1A1-1	9.92%	0.14	529.1	0.0466

Project ID	Specimen ID	Specimen Air voids	FI	Fracture Energy (J/m^2)	(BTU/ft^2)
FAY-35-2.57	1A1-2	10.31%	0.47	869.68	0.0765
FAY-35-2.57	6A1-1	8.06%	0.73	721.18	0.0635
FAY-35-2.57	6A1-2	9.11%	0.12	394.19	0.0347
FRA-70-3.41	11A1-1	1.64%	4.73	1975.62	0.1739
FRA-70-3.41	11A1-2	1.48%	2.23	1851.47	0.1629
FRA-70-3.41	15A1-1	3.93%	3.45	1801.43	0.1585
FRA-70-3.41	15A1-2	3.41%	3.34	1912.54	0.1683
FRA-70-3.41	1A1-1	4.09%	3.90	1928.74	0.1697
FRA-70-3.41	1A1-2	3.09%	5.14	2056.81	0.1810
FRA-70-3.41	9A1-1	1.56%	2.30	1652.18	0.1454
FRA-70-3.41	9A1-2	1.81%	5.30	2405.18	0.2117
FRA-71-5.29	11A2-1	1.76%	2.99	1843.15	0.1622
FRA-71-5.29	11A2-2	2.53%	2.49	1809.79	0.1593
FRA-71-5.29	13A2-1	0.92%	1.81	1857.06	0.1634
FRA-71-5.29	13A2-2	1.08%	1.61	1403.16	0.1235
FRA-71-5.29	1A2-1	2.49%	1.35	1521.52	0.1339
FRA-71-5.29	1A2-2	3.53%	1.81	1698	0.1494
FRA-71-5.29	4A2-1	5.73%	0.66	425.07	0.0374
FRA-71-5.29	4A2-2	5.45%	3.63	874.82	0.0770
FRA-71-5.29	7A1-1	1.96%	0.44	1085.56	0.0955
FRA-71-5.29	7A2-1	3.69%	2.68	1518.66	0.1336
FRA-71-5.29	7A2-2	3.85%	1.29	1405.28	0.1237
FRA-270-21.67	13A1-1	3.31%	5.11	2198.91	0.1935
FRA-270-21.67	13A1-2	3.23%	4.86	2114.87	0.1861
FRA-270-21.67	2A1-1	1.36%	2.22	2495.68	0.2196
FRA-270-21.67	2A1-2	1.00%	2.28	1948.27	0.1714
FRA-270-21.67	7A1-1	1.24%	2.97	1589.49	0.1399
FRA-270-21.67	7A1-2	2.00%	5.18	2149.03	0.1891
FRA-270-35.41 Part 2	12A1-1	7.44%	2.11	749.07	0.0659
FRA-270-35.41 Part 2	12A1-2	8.64%	0.72	803.45	0.0707
FRA-270-35.41 Part 2	5A1-1	3.10%	1.86	1627.47	0.1432
FRA-270-35.41 Part 2	5A1-2	3.62%	0.80	1350.77	0.1189
FRA-270-35.41 Part 2	8A1-1	1.15%	2.47	2046.86	0.1801
FRA-270-35.41 Part 2	8A1-2	2.67%	1.28	1252.47	0.1102
FRA/LIC-161-23.20/0.00	13A1-1	9.96%	0.38	813.33	0.0716
FRA/LIC-161-23.20/0.00	13A1-2	10.20%	1.25	687.77	0.0605
FRA/LIC-161-23.20/0.00	15A1-1	12.24%	2.44	587.22	0.0517
FRA/LIC-161-23.20/0.00	15A1-2	11.32%	1.37	447.86	0.0394
FRA/LIC-161-23.20/0.00	1A1-1	9.32%	0.17	528.14	0.0465

Project ID	Specimen ID	Specimen Air voids	FI	Fracture Energy (J/m^2)	(BTU/ft^2)
FRA/LIC-161-23.20/0.00	1A1-2	9.16%	0.10	471.99	0.0415
FRA/LIC-161-23.20/0.00	4A1-1	10.20%	0.34	557.73	0.0491
FRA/LIC-161-23.20/0.00	4A1-2	9.20%	0.31	1025.14	0.0902
FUL-20-10.86	14A1-1	5.20%	1.50	762.62	0.0671
FUL-20-10.86	14A1-2	4.97%	5.12	650.71	0.0573
FUL-20-10.86	3A1-1	6.04%	3.59	1070.73	0.0942
FUL-20-10.86	3A1-2	5.48%	3.46	1173.99	0.1033
FUL-20-10.86	7A1-1	6.36%	1.65	739.56	0.0651
FUL-20-10.86	7A1-2	6.20%	1.00	778.92	0.0685
GAL-35-8.32	12A1-1	7.42%	1.23	1191.3	0.1048
GAL-35-8.32	12A1-2	6.75%	1.87	1385.27	0.1219
GAL-35-8.32	14A1-1	8.82%	0.55	814.4	0.0717
GAL-35-8.32	14A1-2	7.91%	0.36	1263.15	0.1112
GAL-35-8.32	15A1-1	8.00%	0.32	883.68	0.0778
GAL-35-8.32	15A1-2	8.57%	0.34	1014.01	0.0892
GAL-35-8.32	4A1-1	6.38%	0.25	926.53	0.0815
GAL-35-8.32	4A1-2	7.25%	0.19	813.99	0.0716
GAL-35-8.32	7A1-1	6.63%	2.14	1160.74	0.1021
GAL-35-8.32	7A1-2	7.42%	0.34	1221.39	0.1075
GRE/FAY-35-26.20/0.00	12A1-1	5.21%	0.96	778.9	0.0685
GRE/FAY-35-26.20/0.00	12A1-2	4.89%	1.42	1051.76	0.0926
GRE/FAY-35-26.20/0.00	3A1-1	6.61%	0.25	849.96	0.0748
GRE/FAY-35-26.20/0.00	3A1-2	6.46%	0.16	635.79	0.0559
GRE/FAY-35-26.20/0.00	9A1-1	6.89%	0.37	710.16	0.0625
GRE/FAY-35-26.20/0.00	9A1-2	7.67%	0.89	1070.63	0.0942
HAM-264-6.87	12A1-1	3.02%	0.30	1420.12	0.1250
HAM-264-6.87	12A1-2	3.30%	0.30	1473.14	0.1296
HAM-264-6.87	13A1-1	7.61%	0.87	859.05	0.0756
HAM-264-6.87	13A1-2	7.53%	2.77	1061.86	0.0934
HAM-264-6.87	3A1-1	1.85%	1.08	1486.37	0.1308
HAM-264-6.87	3A1-2	1.93%	1.64	1778.49	0.1565
HAM-264-6.87	9A1-1	3.02%	2.08	1921.97	0.1691
HAM-264-6.87	9A1-2	3.74%	3.27	1777.58	0.1564
HAN/WOO-75-19.22/0.00	14A1-1	5.41%	1.20	1267.3	0.1115
HAN/WOO-75-19.22/0.00	14A1-2	5.45%	1.48	1265.01	0.1113
HAN/WOO-75-19.22/0.00	3A1-1	8.61%	2.38	1404.01	0.1236
HAN/WOO-75-19.22/0.00	3A1-2	8.49%	3.24	1614.48	0.1421
HAN/WOO-75-19.22/0.00	6A1-1	8.84%	1.17	1012.49	0.0891
HAN/WOO-75-19.22/0.00	6A1-2	9.54%	3.31	1289.67	0.1135

Project ID	Specimen ID	Specimen Air voids	FI	Fracture Energy (J/m^2)	(BTU/ft^2)
HAN-30-3.00	12A1-1	7.02%	1.59	747.73	0.0658
HAN-30-3.00	12A1-2	6.70%	1.45	484.81	0.0427
HAN-30-3.00	5A1-1	4.63%	0.70	555.62	0.0489
HAN-30-3.00	5A1-2	4.47%	0.81	510	0.0449
HAN-30-3.00	9A1-1	6.19%	3.60	893	0.0786
HAN-30-3.00	9A1-2	5.76%	3.03	1356.31	0.1194
LAK-2-7.60	10A1-1	7.09%	0.42	634.79	0.0559
LAK-2-7.60	10A1-2	7.80%	0.27	901.17	0.0793
LAK-2-7.60	13A1-1	6.34%	1.69	1618.57	0.1424
LAK-2-7.60	13A1-2	4.84%	2.91	1785.62	0.1571
LAK-2-7.60	16A1-1	5.31%	0.27	992.85	0.0874
LAK-2-7.60	16A1-2	5.87%	0.27	906.96	0.0798
LAK-2-7.60	17A1-1	6.18%	0.66	1165.26	0.1025
LAK-2-7.60	17A1-2	5.63%	0.31	730.43	0.0643
LAK-2-7.60	1A1-1	6.69%	0.30	1391.9	0.1225
LAK-2-7.60	1A1-2	5.35%	1.17	1389.62	0.1223
LAK-2-7.60	5A1-1	2.09%	3.56	2809.84	0.2473
LAK-2-7.60	5A1-2	1.69%	1.46	1512.23	0.1331
LAK-2-7.60	5A2-1	3.90%	3.54	1929.41	0.1698
LAK-2-7.60	5A2-2	3.11%	9.60	2842.39	0.2501
LIC-161-1.83	13A1-1	4.73%	6.18	1674.4	0.1473
LIC-161-1.83	13A1-2	5.05%	8.93	1999.61	0.1760
LIC-161-1.83	3A1-1	5.49%	9.85	2521.01	0.2218
LIC-161-1.83	3A1-2	5.53%	4.95	2153.53	0.1895
LIC-161-1.83	8A1-1	2.91%	8.86	2604.07	0.2292
LIC-161-1.83	8A1-2	3.31%	8.41	2388.98	0.2102
LIC-161-1.83	9A1-1	4.48%	15.76	2457.79	0.2163
LIC-161-1.83	9A1-2	5.09%	8.27	2125.72	0.1871
LUC-2-21.24	13A1-1	4.97%	5.27	1289.93	0.1135
LUC-2-21.24	13A1-2	4.77%	3.91	1193.99	0.1051
LUC-2-21.24	2A1-1	5.28%	2.99	1207.43	0.1063
LUC-2-21.24	2A1-2	4.81%	6.01	1425.55	0.1254
LUC-2-21.24	2A2-1	9.27%	2.45	962.23	0.0847
LUC-2-21.24	2A2-2	8.45%	3.53	1094.05	0.0963
LUC-2-21.24	9A1-1	6.30%	5.03	975.17	0.0858
LUC-2-21.24	9A1-2	4.89%	6.50	1307.27	0.1150
LUC-75-6.70	12A1-1	7.23%	0.28	1051.8	0.0926
LUC-75-6.70	12A1-2	7.04%	0.65	1151.39	0.1013
LUC-75-6.70	2A1-1	6.45%	0.73	1041.54	0.0917

Project ID	Specimen ID	Specimen Air voids	FI	Fracture Energy (J/m^2)	(BTU/ft^2)
LUC-75-6.70	2A1-2	6.76%	0.38	1026.94	0.0904
LUC-75-6.70	7A1-1	6.65%	1.03	1445.76	0.1272
LUC-75-6.70	7A1-2	7.39%	0.95	1310.71	0.1153
MAD-142-0.49	10A1-1	4.33%	2.56	1632.79	0.1437
MAD-142-0.49	10A1-2	4.65%	1.72	998.27	0.0878
MAD-142-0.49	13A1-1	6.37%	3.29	1184.53	0.1042
MAD-142-0.49	13A1-2	6.69%	1.69	871.34	0.0767
MAD-142-0.49	1A1-1	4.61%	0.50	1046.33	0.0921
MAD-142-0.49	1A1-2	5.57%	0.87	1345.46	0.1184
MAD-142-0.49	8A1-1	4.29%	2.05	1611.36	0.1418
MAD-142-0.49	8A1-2	4.00%	0.87	1038.75	0.0914
MED-42-17.68	12A1-1	5.56%	7.22	2562.16	0.2255
MED-42-17.68	12A1-2	4.92%	4.57	1862.75	0.1639
MED-42-17.68	2A1-1	5.81%	7.34	2084.25	0.1834
MED-42-17.68	2A1-2	5.44%	6.21	2266.12	0.1994
MED-42-17.68	6A1-1	3.47%	6.58	2205.04	0.1940
MED-42-17.68	6A1-2	1.94%	4.05	1612.9	0.1419
MED-71-9.56	12A1-1	9.86%	3.63	1519.48	0.1337
MED-71-9.56	12A1-2	10.09%	1.37	795.93	0.0700
MED-71-9.56	1A1-1	10.53%	1.82	696.41	0.0613
MED-71-9.56	1A1-2	10.33%	0.90	574.22	0.0505
MED-71-9.56	6A1-1	9.03%	1.65	831.36	0.0732
MED-71-9.56	6A1-2	8.60%	0.98	853.93	0.0751
MER-219-14.04 "301"	13A1-1	6.08%	1.04	842.2	0.0741
MER-219-14.04 "301"	13A1-2	5.68%	0.71	845.99	0.0744
MER-219-14.04 "301"	6A1-1	7.79%	1.96	752.07	0.0662
MER-219-14.04 "301"	6A1-2	7.87%	4.40	1052.32	0.0926
MER-219-14.04 "301"	8A1-1	6.95%	1.46	793.72	0.0698
MER-219-14.04 "301"	8A1-2	7.75%	0.68	623.89	0.0549
MER-219-14.04 "302"	12B1-1	8.87%	1.76	873.44	0.0769
MER-219-14.04 "302"	12B1-2	8.44%	1.41	511.45	0.0450
MER-219-14.04 "302"	3B1-1	9.59%	0.30	167.67	0.0148
MER-219-14.04 "302"	3B1-2	8.87%	0.24	145.1	0.0128
MER-219-14.04 "302"	3B2-1	7.37%	1.02	511.42	0.0450
MER-219-14.04 "302"	3B2-2	6.85%	2.69	609.66	0.0537
MER-219-14.04 "302"	7B1-1	8.36%	1.70	639.2	0.0562
MER-219-14.04 "302"	7B1-2	8.08%	1.74	521.47	0.0459
MER-219-14.04 "302"	7B2-1	9.59%	2.61	428.74	0.0377
MER-219-14.04 "302"	7B2-2	8.44%	1.69	644.22	0.0567

Project ID	Specimen ID	Specimen Air voids	FI	Fracture Energy (J/m^2)	(BTU/ft^2)
MRW-71-3.17	4A1-1	3.15%	0.34	934.2	0.0822
MRW-71-3.17	4A1-2	3.63%	1.69	1340.1	0.1179
MRW-71-3.17	5A1-1	5.00%	0.30	730.79	0.0643
MRW-71-3.17	5A1-2	4.92%	0.35	952.85	0.0839
MRW-71-3.17	8A1-1	6.62%	0.40	800.63	0.0705
MRW-71-3.17	8A1-2	6.42%	0.32	494.38	0.0435
MRW-71-3.17	8A2-1	6.13%	1.06	605.54	0.0533
MRW-71-3.17	8A2-2	6.21%	0.64	789.16	0.0694
ROS-207-0.00	12A1-1	2.59%	1.07	1772.08	0.1559
ROS-207-0.00	12A1-2	2.47%	0.35	1238.29	0.1090
ROS-207-0.00	14A1-1	3.76%	6.20	2527.78	0.2224
ROS-207-0.00	14A1-2	4.89%	3.56	1841.12	0.1620
ROS-207-0.00	3A1-1	5.62%	1.42	804.05	0.0708
ROS-207-0.00	3A1-2	7.32%	1.42	818.43	0.0720
ROS-207-0.00	6A1-1	5.18%	0.27	916.72	0.0807
ROS-207-0.00	6A1-2	5.26%	0.21	730.79	0.0643
SUM-77-17.20	11A1-1	2.04%	0.81	1378.83	0.1213
SUM-77-17.20	11A1-2	3.19%	0.23	653.37	0.0575
SUM-77-17.20	5A1-1	5.27%	0.52	706.16	0.0621
SUM-77-17.20	5A1-2	4.95%	1.58	1269.21	0.1117
SUM-77-17.20	9A1-1	3.55%	0.70	970.52	0.0854
SUM-77-17.20	9A1-2	4.51%	1.34	1471.65	0.1295
VIN-50-11.75	13A1-1	5.87%	0.68	509.16	0.0448
VIN-50-11.75	13A1-2	6.23%	0.88	744.62	0.0655
VIN-50-11.75	1A1-1	5.06%	0.21	506.11	0.0445
VIN-50-11.75	1A1-2	5.14%	0.22	608.24	0.0535
VIN-50-11.75	9A1-1	5.99%	0.47	564.6	0.0497
VIN-50-11.75	9A1-2	5.83%	0.41	633.45	0.0557
WAR-71-3.78	10A1-1	6.85%	0.31	808.13	0.0711
WAR-71-3.78	10A1-2	5.50%	0.25	547.95	0.0482
WAR-71-3.78	2A1-1	3.51%	1.07	1089.77	0.0959
WAR-71-3.78	2A1-2	2.43%	3.10	1598.79	0.1407
WAR-71-3.78	5A1-1	2.07%	2.02	1680.31	0.1479
WAR-71-3.78	5A1-2	1.95%	1.81	1346.91	0.1185
WAR-71-14.20	14A1-1	7.02%	2.31	1453.15	0.1279
WAR-71-14.20	14A1-2	6.58%	3.96	1244.67	0.1095
WAR-71-14.20	2A1-1	5.39%	8.16	2113.82	0.1860
WAR-71-14.20	2A1-2	5.55%	5.87	1779.57	0.1566
WAR-71-14.20	6A1-1	6.38%	3.67	1661.61	0.1462

Project ID	Specimen ID	Specimen Air voids	FI	Fracture Energy (J/m²)	(BTU/ft²)
WAR-71-14.20	6A1-2	6.50%	3.91	1825.58	0.1607
WAY-30-11.86 "301"	C3-1	10.75%	0.20	472.54	0.0416
WAY-30-11.86 "301"	C3-2	9.83%	0.14	369.23	0.0325
WAY-30-11.86 "301"	C8-1	8.99%	0.12	367.6	0.0323
WAY-30-11.86 "301"	C8-2	9.47%	0.18	467.4	0.0411
WAY-30-11.86 "302"	10A1-1	6.96%	0.27	949.09	0.0835
WAY-30-11.86 "302"	10A1-2	6.60%	0.69	1032.61	0.0909
WAY-30-11.86 "302"	10A2-1	8.57%	0.66	704.58	0.0620
WAY-30-11.86 "302"	10A2-2	8.01%	1.06	886.85	0.0780
WAY-30-11.86 "302"	2A1-1	4.47%	0.27	735.13	0.0647
WAY-30-11.86 "302"	2A1-2	4.83%	0.30	651.1	0.0573
WAY-30-11.86 "302"	8A1-1	4.39%	0.26	867.82	0.0764
WAY-30-11.86 "302"	8A1-2	3.74%	0.44	1093.94	0.0963
WOO-75-2.37	12A1-1	7.30%	1.55	1271.57	0.1119
WOO-75-2.37	12A1-2	5.93%	1.01	1475.28	0.1298
WOO-75-2.37	1A1-1	6.12%	0.88	958.96	0.0844
WOO-75-2.37	1A1-2	4.51%	0.44	1013.33	0.0892
WOO-75-2.37	8A1-1	5.81%	0.82	1141.25	0.1004
WOO-75-2.37	8A1-2	6.59%	1.96	981.87	0.0864
WOO-75-19.43	10A1-1	4.53%	1.10	1592.25	0.1401
WOO-75-19.43	10A1-2	4.69%	1.14	2088.44	0.1838
WOO-75-19.43	13A1-1	9.03%	1.62	973.06	0.0856
WOO-75-19.43	13A1-2	8.68%	0.66	624.45	0.0550
WOO-75-19.43	4A1-1	3.79%	0.38	932.86	0.0821
WOO-75-19.43	4A1-2	4.22%	1.90	1759.48	0.1548
WOO-75-10.61	10A1-1	8.83%	0.82	669.15	0.0589
WOO-75-10.61	10A1-2	7.26%	0.90	708.61	0.0624
WOO-75-10.61	1A1-1	8.56%	1.68	614.42	0.0541
WOO-75-10.61	1A1-2	7.89%	3.90	1108.01	0.0975
WOO-75-10.61	6A1-1	6.99%	4.71	1470.01	0.1294
WOO-75-10.61	6A1-2	6.83%	3.35	1377.62	0.1212
WOO-795-2.01	10A1-1	5.90%	8.50	1198.07	0.1054
WOO-795-2.01	10A1-2	5.42%	4.38	885.29	0.0779
WOO-795-2.01	2A1-1	5.30%	2.54	702.79	0.0618
WOO-795-2.01	2A1-2	5.26%	3.41	818.05	0.0720
WOO-795-2.01	5A1-1	5.70%	3.00	831.66	0.0732
WOO-795-2.01	5A1-2	5.94%	3.26	795.18	0.0700

Appendix K: TSR Test Results

Project ID (County-Route-Section)	Region	District	Project #	PID	Spec Year	Plan Base Type	Plan Base Thickness (in)	Plan Base Thickness (mm)	Mix Type Used	Aggregate Type Used	RAP Used	RAS Used	Pavement Age (yr)	Performance	Measured/Predicted ADTT	Unconditioned				Conditioned				TSR
																No. of Specimens	Average Tensile Strength (psi)	Average Tensile Strength (kPa)	Average Air Voids	No. of Specimens	Average Tensile Strength (psi)	Average Tensile Strength (kPa)	Average Air Voids	
ASD-39-0.00	NE	3	2004-0500	23578	2002	302, APP	8	203	302	Blend	Y	N	12	Poor	1.32	3	173.81	1198.33	8.75%	3	92.87	640.28	8.77%	0.53
ATH-33-11.74	SE	10	2017-0008	84468	2016	302	5.5	140	302	Blend	Y	N	0	N/A	N/A	3	236.56	1630.98	1.90%	4	187.75	1294.50	1.83%	0.79
BUT-75-5.91	SW	8	2008-0246	75971	2005	302	9	229	302	Blend	Y	N	7	Avg.	0.77	3	241.60	1665.72	4.85%	3	189.05	1303.43	4.73%	0.78
CLA-70-13.98	SW	7	2010-0243	84664	2008	302	10.5	267	302	Blend	Y	N	4	Avg.	0.89	3	213.86	1474.52	3.31%	3	216.08	1489.77	3.20%	1.01
CLI-73-6.52	SW	8	2009-0244	78571	2008	880	11	279	301	LMS	Y	N	5	Avg.	0.30	3	226.41	1561.05	7.25%	3	167.25	1153.16	6.79%	0.74
CLI-73-12.03	SW	8	2006-0413	78569	2005	880	10.5	267	302	LMS	Y	N	9	Avg.	0.25	3	97.77	674.10	5.21%	4	77.47	534.14	5.63%	0.79
COS-36-20.83	C	5	1996-0278	14142	1995	302	8	203	302	Blend	UNK	N	19	Avg.	0.56	3	246.13	1697.01	6.87%	3	61.89	426.70	6.76%	0.25
CUY-71-0.00	NE	12	1998-0748	15717	1997	302	11.3	286	302	LMS	Y	N	18	Exc.	0.93	3	134.56	927.72	8.06%	3	115.38	795.51	7.98%	0.86
CUY/SUM-77-0.00/32.73	NE	12	2016-3019	79671	2013	302	7.5	191	302	UNK	Y	N	0	N/A	N/A	3	159.61	1100.48	3.22%	3	111.47	768.55	3.13%	0.70
DEF-24-7.96	NW	1	2006-0087	24337	2005	880	11	279	302	LMS	N	Y	9	Avg.	UNK	3	141.18	973.42	7.03%	3	45.31	312.43	7.11%	0.32
DEL-23-17.64	C	6	2012-0284	79370	2010	302	4	102	302	LMS	Y	N	5	N/A	N/A	3	214.55	1479.24	6.29%	3	187.39	1292.01	6.45%	0.87
DEL-23-19.24	C	6	1997-0335	16350	1995	302	12	305	302	LMS	N	N	20	N/A	0.77	3	169.21	1166.62	8.43%	3	95.47	658.24	8.45%	0.56
FAI-33-7.31	C	5	2001-0136	16293	1997	880	9	229	302	Blend	Y	N	14	Poor	0.90	3	204.37	1409.04	10.08%	3	119.83	826.18	9.88%	0.59
FAI-33-13.25	C	5	2002-0110	16294	1997	880	8	203	301	GRVL	UNK	N	14	Avg.	0.70	3	197.92	1364.61	7.22%	3	86.94	599.44	7.16%	0.44
FAI-33-17.44 SR 13	C	5	2002-0446	16295	1997	301	6	152	301	GRVL	Y	N	13	N/A	UNK	3	285.22	1966.47	4.66%	4	189.19	1304.37	4.39%	0.66
FAI-33-17.44	C	5	2002-0446	16295	1997	880	7	178	302	Blend	Y	N	13	Avg.	2.36	3	229.27	1580.74	5.95%	3	121.45	837.34	5.84%	0.53
FAI-33-19.79 SR 18	C	5	2003-0046	23057	1997	301	6	152	301	GRVL	Y	N	12	N/A	UNK	3	253.70	1749.16	8.30%	3	162.11	1117.72	8.15%	0.64
FAI-33-19.79	C	5	2003-0046	23057	1997	880	7	178	302	LMS	Y	N	12	Avg.	2.36	3	222.24	1532.30	5.84%	3	152.94	1054.46	5.76%	0.69
FAY-35-2.57	C	6	2000-0577	9078	1997	880	12	305	UNK	LMS	Y	N	16	Avg.	9.35	3	167.04	1151.71	6.72%	4	115.79	798.31	5.97%	0.69
FRA-70-3.41	C	6	2015-0396	25594	2013	302	12	305	302	UNK	Y	N	0	N/A	N/A	3	166.60	1148.63	2.45%	3	129.56	893.27	2.30%	0.78
FRA-71-5.29	C	6	2015-0395	84868	2013	302	12	305	302	Blend	Y	N	0	N/A	N/A	3	200.77	1384.26	1.90%	3	186.39	1285.13	1.66%	0.93
FRA/LIC-161-23.20/0.00	C	6	2006-0150	24486	2005	880	7.5	191	301	LMS	Y	N	9	Avg.	0.82	3	163.01	1123.87	9.12%	3	83.82	577.88	9.00%	0.51
FRA-270-21.67	C	6	2015-0249	81747	2013	302	10	254	302	Blend	Y	N	0	N/A	N/A	3	157.71	1087.34	1.65%	4	113.94	785.60	1.22%	0.72
FRA-270-35.41 Part 2	C	6	2017-0003	84620	2016	302	10.5	267	302	Blend	Y	N	0	N/A	N/A	3	203.32	1401.83	6.29%	3	125.82	867.46	6.10%	0.62
FUL-20-10.86	NW	2	2000-0341	19342	1997	302	12	305	302	LMS	Y	N	17	Avg.	0.64	3	130.51	899.83	5.50%	3	71.93	495.91	5.46%	0.55
GAL-35-8.32	SE	10	2007-0334	22520	2005	301	8	203	301	GRVL	N	N	10	N/A	N/A	3	196.88	1357.41	7.42%	4	117.39	809.37	7.65%	0.60
GRE/FAY-35-26.20/0.00	C	6	2000-0091	04388	1997	301	11	279	UNK	LMS	Y	UNK	16	Exc.	2.49	3	176.43	1216.39	5.61%	3	113.60	783.22	5.46%	0.64

Project ID (County-Route-Section)	Region	District	Project #	PID	Spec Year	Plan Base Type	Plan Base Thickness (in)	Plan Base Thickness (mm)	Mix Type Used	Aggregate Type Used	RAP Used	RAS Used	Pavement Age (yr)	Performance	Measured/Predicted ADTT	Unconditioned No. of Specimens	Unconditioned Average Tensile Strength (psi)	Unconditioned Average Tensile Strength (kPa)	Unconditioned Average Air Voids	Conditioned No. of Specimens	Conditioned Average Tensile Strength (psi)	Conditioned Average Tensile Strength (kPa)	Conditioned Average Air Voids	TSR
HAM-264-6.87	SW	8	2000-0135	13853	1997	301/ 302	3 in/ 8 in	75 mm/ 200 mm	UNK	UNK	UNK	UNK	17	N/A	0.32	3	219.33	1512.21	3.33%	3	140.15	966.30	3.26%	0.64
HAN/WOO-75-19.22/0.00	NW	2	2014-3000	95437	(-)	302	12.5	318	302	LMS	Y	Y	0	N/A	N/A	3	158.37	1091.88	7.12%	3	114.20	787.40	6.73%	0.72
HAN-30-3.00	NW	1	2005-0003	77302	2002	880	15.4	390	302	LMS	Y	Y	10	Avg.	1.10	3	85.74	591.11	6.49%	3	53.91	371.66	6.17%	0.63
LAK-2-7.60	NE	12	2010-0215	79545	2008	302	10	254	302	LMS	Y	N	5	Avg.	0.50	3	213.04	1468.81	4.67%	3	194.08	1338.09	7.22%	0.91
LIC-161-1.83	C	5	2016-8030	97879	2016	301	7	178	301	LMS	Y	N	0	N/A	N/A	3	161.97	1116.75	3.86%	3	114.52	789.58	3.76%	0.71
LUC-2-21.24	NW	2	1999-0141	9159	1997	301	10	254	302	LMS	Y	N	16	Exc.	0.37	3	93.62	645.48	7.34%	3	89.07	614.09	6.99%	0.95
LUC-75-6.70	NW	2	2014-0536	76032	2013	302	11.5	292	302	LMS	Y	N	0	N/A	N/A	3	189.48	1306.38	6.88%	3	152.48	1051.27	6.80%	0.80
MAD-142-0.49	C	6	2001-0317	11739	1997	301	8	203	301	LMS	UNK	N	14	Avg.	0.54	3	207.51	1430.74	5.15%	3	99.97	689.23	5.13%	0.48
MED-42-17.68	NE	3	2016-0430	92954	2013	301	6	152	UNK	UNK	UNK	UNK	0	N/A	N/A	3	142.57	982.95	4.81%	3	123.01	848.09	4.41%	0.86
MED-71-9.56	NE	3	2005-0343	14018	2002	302	10.5	267	UNK	UNK	Y	N	10	Avg.	0.70	3	147.99	1020.37	8.89%	3	63.87	440.38	8.70%	0.43
MER-219-14.04 "302"	NW	7	2005-0313	19968	2002	302	7.5	191	302	LMS	N	N	12	Avg.	0.61	3	137.28	946.52	5.11%	3	113.24	780.78	5.06%	0.82
MER-219-14.04 "301"	NW	7	2005-0313	19968	2002	301	3	76	301	LMS	N	UNK	12	Avg.	0.61	3	160.46	1106.31	8.09%	3	103.86	716.05	7.96%	0.65
MRW-71-3.17	C	6	2013-3001	86920	2010	302	11	279	UNK	UNK	UNK	UNK	1	Avg.	0.55	3	218.24	1504.66	4.65%	3	197.80	1363.74	4.40%	0.91
ROS-207-0.00	SW	9	2004-0533	18492	2002	880	8.5	216	301	GRVL	Y	N	11	Exc.	0.35	3	173.63	1197.11	4.89%	3	160.99	1109.95	4.56%	0.93
SUM-77-17.20	NE	4	2006-0151	16514	2005	880	9.5	241	302	LMS	Y	N	9	Exc.	0.50	3	206.22	1421.79	3.90%	3	131.51	906.75	3.78%	0.64
VIN-50-11.75	SE	10	2001-0123	10504	1997	301	8	203	301	GRVL	N	N	16	Poor	1.77	3	216.72	1494.20	5.98%	4	133.99	923.82	6.18%	0.62
WAR-71-3.78	SW	8	1999-0780	10696	1997	302	11.8	300	UNK	UNK	UNK	UNK	16	Avg.	0.40	3	131.85	909.06	3.13%	3	167.38	1154.05	2.82%	1.27
WAR-71-14.20	SW	8	2010-0280	22950	2008	880	11.3	286	302	Blend	Y	N	2	N/A	UNK	3	117.53	810.34	5.54%	4	109.94	758.03	6.05%	0.94
WAY-30-11.86 "301"	NE	3	2004-0044	16285/ 16287	2002	301, APP	3	76	301	LMS	Y	N	11	N/A	0.87	2	215.37	1484.91	10.61%	2	133.71	921.86	10.15%	0.62
WAY-30-11.86 "302"	NE	3	2004-0044	16285/ 16287	2002	302	9/4 FRL		302	LMS	Y	N	11	Poor	0.87	3	176.14	1214.41	4.79%	3	139.50	961.78	4.76%	0.79
WOO-75-10.61	NW	2	2014-0170	95435	2013	302	11	279	302	LMS	Y	N	0	N/A	N/A	3	149.89	1033.44	6.57%	3	117.24	808.34	6.35%	0.78
WOO-75-19.43	NW	2	2014-0237	25521	2013	301	13	330	UNK	LMS	Y	N	0	N/A	N/A	3	238.61	1645.15	5.50%	3	193.02	1330.84	5.38%	0.81
WOO-75-2.37	NW	2	2014-0199	95436	2013	302	11	279	302	LMS	Y	UNK	0	N/A	N/A	3	144.56	996.66	4.98%	3	112.76	777.45	4.80%	0.78
WOO-795-2.01	NW	2	1999-0505	13725	1997	302	9.8	250	302	LMS	Y	N	17	Poor	0.14	3	138.11	952.19	4.30%	3	100.93	695.87	4.28%	0.73

Appendix L: Cantabro Mass Loss Test Results

Project ID	Specimen 1		Specimen 2		Average Mass Loss
	Air Voids	Mass Loss	Air Voids	Mass Loss	
ASD-39-0.00	8.42%	24.16%	9.07%	36.10%	30.13%
ATH-33-11.74	3.43%	14.79%	3.35%	11.64%	13.21%
BUT-75-5.91	5.31%	17.49%	5.63%	22.44%	19.96%
CLI-73-12.03	4.50%	7.60%	5.30%	10.38%	8.99%
CLI-73-6.52	7.64%	22.36%	6.52%	13.15%	17.75%
COS-36-20.83	7.03%	95.36%	6.79%	96.02%	95.69%
CUY/SUM-77-0.00/32.73	1.58%	6.30%	1.82%	5.65%	5.97%
CUY-71-0.00	9.87%	59.76%	10.39%	52.02%	55.89%
DEF-24-7.96	6.68%	93.45%	7.08%	90.51%	91.98%
DEL-23-17.64	3.88%	13.56%	6.68%	27.90%	20.73%
DEL-23-19.24	7.90%	43.43%	10.88%	94.72%	69.07%
FAI-33-13.25	2.71%	89.27%	8.12%	50.31%	69.79%
FAI-33-17.44	4.90%	89.85%	4.38%	25.52%	57.69%
FAI-33-17.44 SR 13	5.18%	18.01%	3.86%	18.96%	18.49%
FAI-33-19.79	3.51%	16.61%	8.26%	55.21%	35.91%
FAI-33-19.79 SR 18	7.38%	41.38%	7.95%	75.20%	58.29%
FAI-33-7.31	10.08%	43.39%	8.06%	33.90%	38.64%
FAY-35-2.57	4.30%	27.75%	3.99%	33.66%	30.71%
FRA/LIC-161-23.20/0.00	10.04%	77.12%	10.48%	90.67%	83.90%
FRA-270-21.67	3.71%	7.12%	2.00%	8.26%	7.69%
FRA-270-35.41 Part 2	4.85%	11.12%	5.89%	20.10%	15.61%
FRA-70-3.41	1.81%	6.90%	3.81%	8.24%	7.57%
FRA-71-5.29	4.93%	16.36%	1.72%	6.79%	11.57%
FUL-20-10.86	4.85%	26.90%	5.80%	35.71%	31.31%
GAL-35-8.32	6.26%	54.70%	7.08%	43.64%	49.17%
GRE/FAY-35-26.20/0.00	5.60%	32.32%	6.14%	48.57%	40.45%
HAM-264-6.87	3.30%	14.35%	4.35%	20.05%	17.20%
HAN/WOO-75-19.22/0.00	6.78%	10.94%	10.75%	74.23%	42.59%
HAN-30-3.00	5.41%	27.02%	6.78%	95.34%	61.18%
LAK-2-7.60	6.81%	17.72%	-	51.17%	34.45%
LIC-161-1.83	4.6%	23.48%	1.9%	7.12%	15.30%
LUC-2-21.24	6.2%	39.99%	7.4%	39.24%	39.61%
LUC-75-6.70	6.9%	15.64%	7.0%	10.64%	13.14%
MAD-142-0.49	4.9%	15.69%	2.4%	11.53%	13.61%
MED-42-17.68	4.7%	6.00%	2.1%	5.16%	5.58%
MED-71-9.56	7.3%	10.77%	9.4%	44.64%	27.70%
MER-219-14.04 "301"	12.8%	22.86%	8.3%	44.88%	33.87%

Project ID	Specimen 1		Specimen 2		Average Mass Loss
	Air Voids	Mass Loss	Air Voids	Mass Loss	
MER-219-14.04 "302"	6.0%	27.79%	7.3%	45.53%	36.66%
ROS-207-0.00	4.8%	19.86%	4.2%	18.54%	19.20%
SUM-77-17.20	3.0%	9.37%	4.2%	12.59%	10.98%
VIN-50-11.75	6.4%	29.22%	6.2%	42.14%	35.68%
WAR-71-14.20	8.0%	42.06%	7.5%	11.90%	26.98%
WAR-71-3.78	2.3%	11.74%	2.3%	19.60%	15.67%
WOO-75-10.61	5.1%	16.32%	5.8%	11.32%	13.82%
WOO-75-19.43	5.7%	24.61%	10.2%	81.09%	52.85%
WOO-75-2.37	6.4%	8.91%	4.6%	8.57%	8.74%

Appendix M: Use of Pavement ME Design Software to Evaluate Ohio's AC Base Mixtures

Introduction

A mixture evaluation procedure was used to predict a dense-graded asphalt concrete (AC) mixture's resistance to area fatigue cracking and rutting in accordance with the Mechanistic-Empirical Pavement Design Guide (MEPDG) software. The evaluation procedure estimates the fracture and permanent deformation constants from mixture volumetric parameters, similar to the regression equations used to calculate dynamic modulus, and was formalized under NCHRP Project 1-40B when fundamental fracture fatigue strength and repeated load plastic deformation tests were unavailable. The purpose of this appendix is to evaluate the different AC base mixtures using the Pavement ME Design software by adjusting the transfer function coefficients that are volumetric dependent using the job mix formulas.

Appendix M is grouped into four parts. The first two parts provide an overview of the recommended procedure for making adjustments to the plastic deformation and fatigue cracking transfer function coefficients that were determined under NCHRP project 1-37A. The transfer function coefficients are mixture independent for both fatigue cracking and rutting. The third part segregates the AC base mixtures included in the project into three categories based on the predicted performance estimated from the job mix formula data with different mixture properties. The fourth part is a comparison of the mixture test results performed on the AC base mixtures with the different groups of performance.

Estimating the Plastic Strain Coefficients from Mixture Volumetric Properties

The rut depth prediction equation in the ME Design software to predict rutting over time is given below.

$$\frac{\varepsilon_p}{\varepsilon_r} = \beta_{r1}(10)^{k_{r1}}(T)^{\beta_{r2}(k_{r2})}(N)^{\beta_{r3}(k_{r3})} \tag{M.1}$$

Where:

T = Temperature, ˚F
N = Number of load applications for a specific temperature range or season.
k_{r1} = Global calibration coefficient = -3.4488, for all HMA mixes.
k_{r2} = Global calibration coefficient = 1.5606, for all HMA mixes.
k_{r3} = Global calibration coefficient = 0.4791, for all HMA mixes.
$\beta_{r1,r2,r3}$ = Local calibration coefficients for rutting (for Option A, all = 1.0).

Differences in volumetric properties are taken into account through changes in the dynamic modulus. It has been found, however, that AC mixtures with similar dynamic modulus values can have significantly different permanent deformation constants and resulting rut depths. The following lists the steps used to revise the above permanent deformation equation to account for volumetric differences that are known to have a significant effect on the permanent deformation constants. This procedure follows the method documented in NCHRP Report 719.

Intercept of Transfer Function

The first step is to determine the gradation index for each HMA mixture, which is a value that determines how close the aggregate blend or gradation of the mixture is to the theoretical maximum density line. The gradation index is defined as the absolute difference between the actual gradation and the maximum density line (FHWA 0.45 power gradation chart) using sieve sizes 3/8 in (9.5 mm), #4 (4.75 mm), #8 (2.36 mm), #16 (1.18 mm), #30 (0.6 mm), and #50 (0.3 mm). The reason for using these sieve sizes is that they can have an impact on the Voids in Mineral Aggregate (VMA) – increases in fines produced during construction within this range can collapse the VMA on some mixtures. The gradation index is used to refine the adjustment factors for predicting rutting.

$$GI = \sum_{i=3/8}^{\#50} \left| P_i - P_{i(0.45)} \right| \tag{M.2}$$

Where:

GI = Gradation Index

P_i = Percent passing sieve i, %

$P_{i(0.45)}$ = Percent passing sieve i for the FHWA 0.45 maximum density line, %

The field-adjusted, laboratory-derived intercept is required to estimate the transfer function intercept for each transfer function. The recommended relationships to estimate the laboratory-derived intercept from the secondary region of triaxial tests is provided below.

$$I_{Triaxial} = 10^{-3.6} \left(\frac{V_a}{V_{Design}} \right)^{0.52} \left(Log(VFA) \right) \left(F_{Index} \right) \left(C_{Index} \right) \tag{M.3}$$

Where:

V_a = In-place air voids of the HMA layer, percent.

V_{Design} = Design air void level for selecting the target asphalt content, percent.

VFA = Voids Filled with Asphalt, percent.

F_{Index} = An index number related to the fine aggregate angularity (FAA) of the combined aggregate blend; refer to the top portion of Table 11–27 for the recommended values. The FAA value is usually measured during mixture design and for aggregate source approval (AASHTO T 304).

C_{Index} = An index number related to the coarse aggregate angularity (CAA) of the combined aggregate blend; refer to the bottom portion of Table 11–27 for the recommended values. The coarse aggregate angularity of the combined coarse aggregate of a mixture is measured in the laboratory (AASHTO T 326), and most agencies do have required limits for the minimum amount of coarse aggregate with two crushed faces for varying truck volumes (AASHTO TP 61).

Table 11–27. Aggregate Properties for Determining the Mixture Adjustment Factors.

<table>
<tr><th rowspan="2">Fine Aggregate</th><th rowspan="2">Gradation</th><th colspan="5">Fine Aggregate Angularity; AASHTO T 304</th></tr>
<tr><th colspan="2"><45</th><th colspan="3">>45</th></tr>
<tr><td rowspan="2">FAA Index Value</td><td>External to Restricted Zone</td><td colspan="2">1.0</td><td colspan="3">0.9</td></tr>
<tr><td>Through Restricted Zone</td><td colspan="2">1.05</td><td colspan="3">1.0</td></tr>
<tr><th rowspan="2">Coarse Aggregate</th><th rowspan="2">Gradation</th><th colspan="5">Percentage Coarse Aggregate with Two Crushed Faces; AASHTO TP 61</th></tr>
<tr><th>0</th><th>25</th><th>50</th><th>75</th><th>100</th></tr>
<tr><td rowspan="2">CAA Index Value</td><td>Well Graded</td><td>1.1</td><td>1.05</td><td>1.0</td><td>1.0</td><td>0.9</td></tr>
<tr><td>Gap Graded</td><td>1.2</td><td>1.1</td><td>1.05</td><td>1.0</td><td>0.9</td></tr>
</table>

The design air void content is determined from mixture design charts (air voids as a function of asphalt content), and is the air void content at the target asphalt content (or the value expected during production of the mixture). Figure 11-50 shows an example in determining this value or parameter for a specific mixture. The accuracy of this parameter is dependent on how close the laboratory compaction effort simulates the field compaction that occurs under the rollers and truck traffic. The intercept value from equation M.3 is entered in Figure 11-51 to estimate the field matched intercept for the Kaloush transfer function.

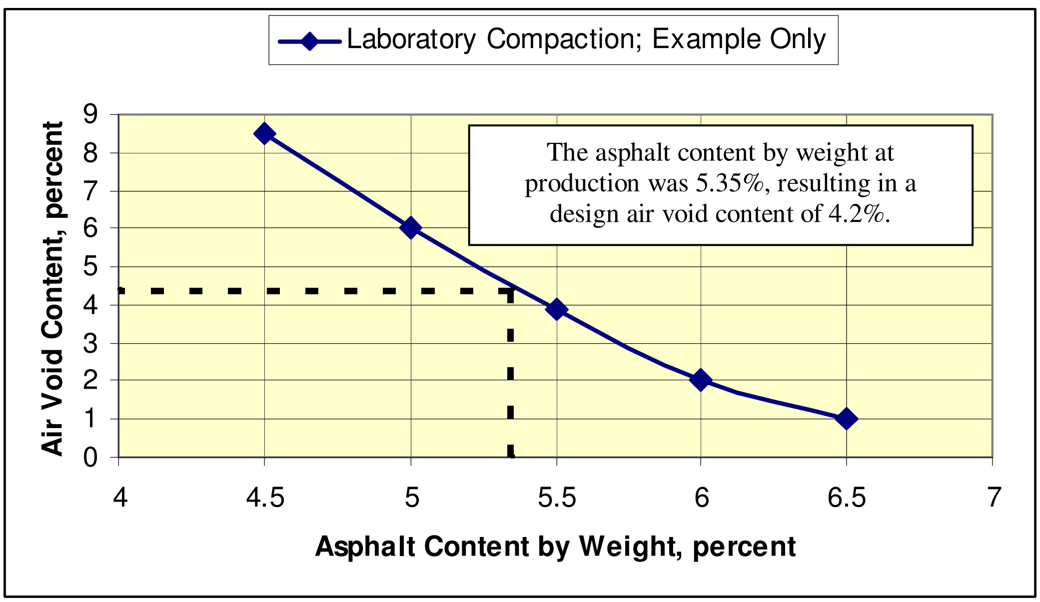


Figure 11-50. Graphical Example Determining the Design Air Void Content from the Laboratory Mixture Design Chart.

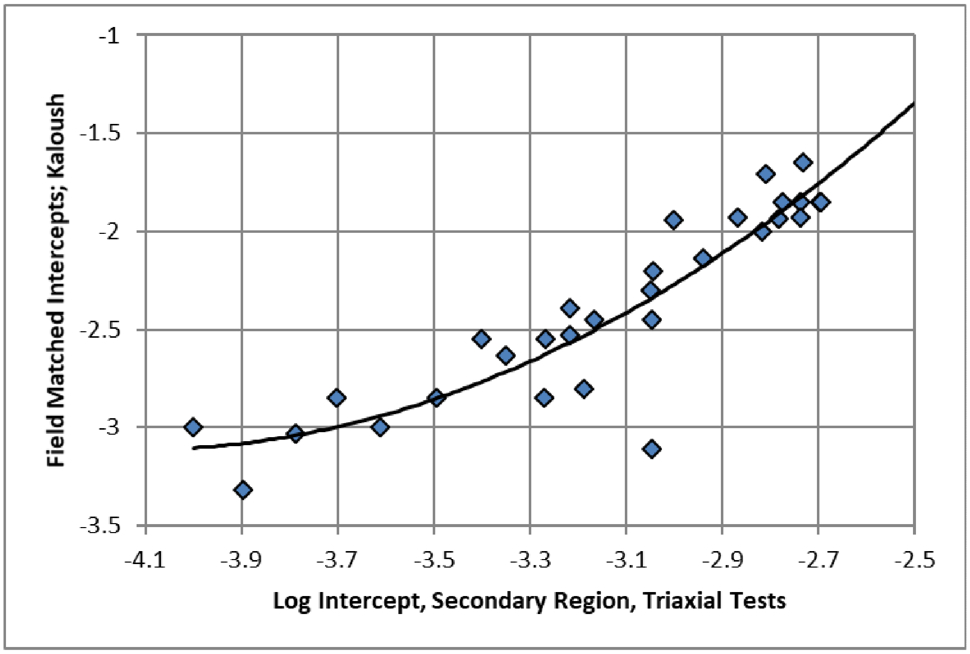


Figure 11-51. Determining the Field Matched Intercept from Laboratory-Derived Values from Repeated Load Triaxial Tests, Kaloush-Witczak Transfer Function.

m-Value of Transfer Functions

The relationship for estimating the m-value for dense-graded designed aggregate blends is provided in equation M.4:

$$m - Value_{Neat} = 0.265\left(\frac{P_b}{P_{b(Opt)}}\right)^{0.75} \tag{M.4}$$

Where:

P_b = Asphalt content by weight at construction (the in-place value), percent.
$P_{b(Sat)}$ = Saturation or optimum asphalt content by weight, percent. This parameter defines the asphalt content at which the VMA starts to increase or the density of the mixture starts to decrease.

The saturation asphalt content by weight parameter is determined from the mixture design charts (mixture density as a function of asphalt content), and is the asphalt content where the density begins to significantly decrease or where the VMA begins to significantly increase. This value is determined in the laboratory and is not a well-defined parameter. Figure 11-52a shows an example of a sensitive HMA mixture in determining this value, while Figure 11-52b shows an example for a non-sensitive mixture.

For the use of modified asphalts, the m-value for neat asphalt mixtures is adjusted by Equation M.5:

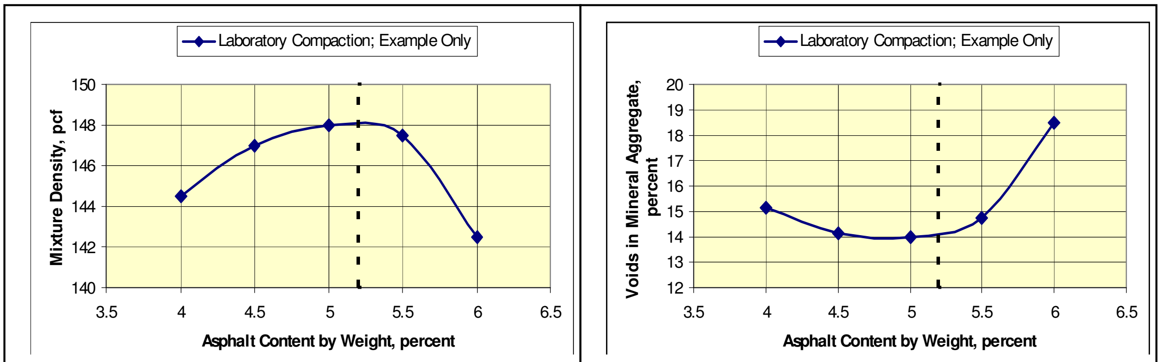
$$m - Value_{Modified} = m_b\left(m - Value_{Neat}\right) \tag{M.5}$$

Where:

m_b = An adjustment that accounts for the use of modified mixtures for the same aggregate blend of neat asphalt mixtures and defined below.

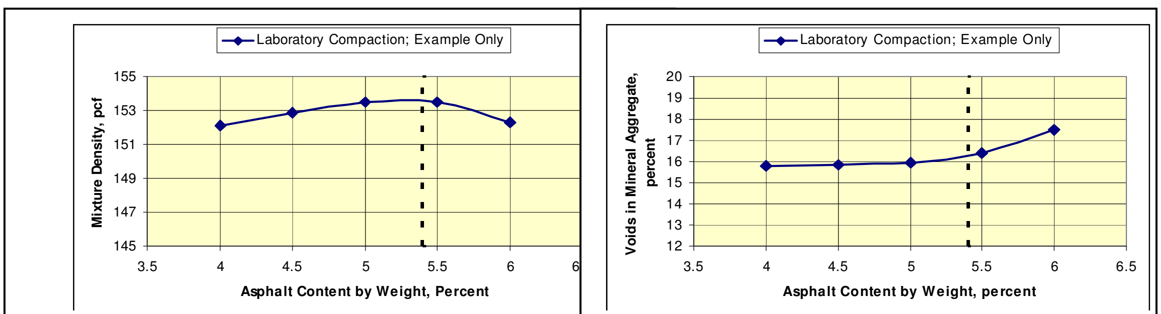
For m-values less than or equal to 0.2: $m_b = 1.0$.

For m-Values greater than 0.2: $m_b = 0.072 + (m - Value)0.64$ (M.6)



The saturation asphalt content is defined as 5.2% by weight for this example, because the mixture density begins to significantly decrease and the VMA starts to increase at about that value.

a.) Example for a Sensitive Mixture



The saturation asphalt content is defined as 5.4% by weight for this example, because the VMA starts to increase at about that value. The HMA density also starts to decrease at asphalt content slightly above that value.

b.) Example for a Non-Sensitive Mixture

Figure 11-52. Graphical Example: Determining the Saturation Asphalt Content from the Laboratory Mixture Design Chart (1 pcf = 16 kg/m^3).

The next step is to convert the laboratory-derived slope of the secondary region of the plastic deformation test results, the m-value, to a field-adjusted plastic deformation exponent, the k_{r3}-value, of the number of axle loads (N-term in the transfer function using Figure 11-53.

Temperature Term Exponent of Kalouš Transfer Function

The Kaloush transfer function is the only one of the three recommended for use which includes temperature as a dependent variable. The temperature exponent is set to 1.5606.

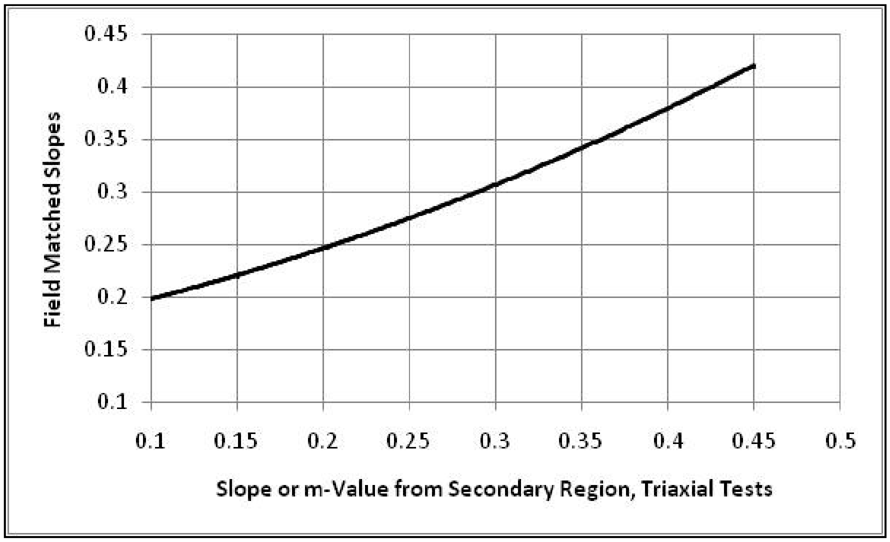


Figure 11-53. Determining the Field Matched Slopes from Laboratory-Derived Values from Repeated Load Triaxial Tests, Kaloush-Witczak Transfer Function.

Estimating Fatigue Fracture Coefficients from Volumetric Properties

The load related cracking equation is included in the ME Design software to predict bottom-up cracking over time.

$$FC_{Bottom-UP} = \left(\frac{C_4}{1 + e^{\left(C_1 C_1^{'} + C_2 C_c^{'} \left(Log_{10} D100\right)\right)}} \right) \frac{1}{60} \tag{M.7}$$

Where:

C_1, C_2 = 1.0
C_1' = $-2C_2'$
C_2' = $-2.40874-39.748(1-h_{ac})^{-2.856}$
C_4 = 6,000
h_{ac} = Thickness of the HMA layer, inches

$$D = DamageIndex = \sum \frac{n_{actual}}{N_{Allowable}} \tag{M.8}$$

$$N_{Allowable} = k_{f1} \left(\beta_{f1}\right) k_1^{'} \left(\frac{1}{\varepsilon_t} \right)^{k_{f2}\beta_{f2}} \left(\frac{1}{E_{ac}} \right)^{k_{f3}\beta_{f3}} \tag{M.9}$$

$$k_{1Bottom-Up}^{'} = \frac{1}{0.000398 + \left(\frac{0.003602}{1 + e^{(11.02 - 3.49 h_{ac})}} \right)} \tag{M.10}$$

$$C = 10^M \tag{M.11}$$

$$M = 4.84 \left(\frac{V_{be}}{V_a + V_{be}} \right) - 0.69 \tag{M.12}$$

k_{f1} = Global calibration coefficient = 0.00432
k_{f2} = Global calibration coefficient = 3.9492
k_{f3} = Global calibration coefficient = 1.281
$\beta_{f1}, \beta_{f2}, \beta_{f3}$ = Local calibration factors for fatigue cracking; all equal 1.0 for global calibration.

Differences in the fracture characteristics of mixtures are taken into account in the ME Design software through changes in the dynamic modulus and volumetric properties (air voids and asphalt content). However, the residual error term for bottom-up cracking was found to be high. The following lists the steps used to revise the above fatigue or load related cracking coefficients to increase the accuracy of the transfer function (see Equation M.7 and Equation M.9).

Estimate the Fracture Parameters: k_{f1} and k_{f3}

The k_{f1} parameter in equation M.9 is adjusted based on the voids filled with asphalt (VFA) for the AC base layer, refer to Figure 11-54. This value is used to replace the global calibration factor included in the ME Design software. The k_{f3} parameter in equation M.9 is related to and estimated from the k_{f1} parameter for the AC base layer that is determined using Figure 11-55. This value (k_{f3}) is used to replace the global calibration factor included in the ME Design software.

Estimate the Fatigue Cracking Transfer Function Field Adjusted Parameter

The C_2 coefficient in equation M.7 is based on the VFA of the AC base layer, refer to Figure 11-56. This value is used to replace the global calibration factor included in the ME Design software.

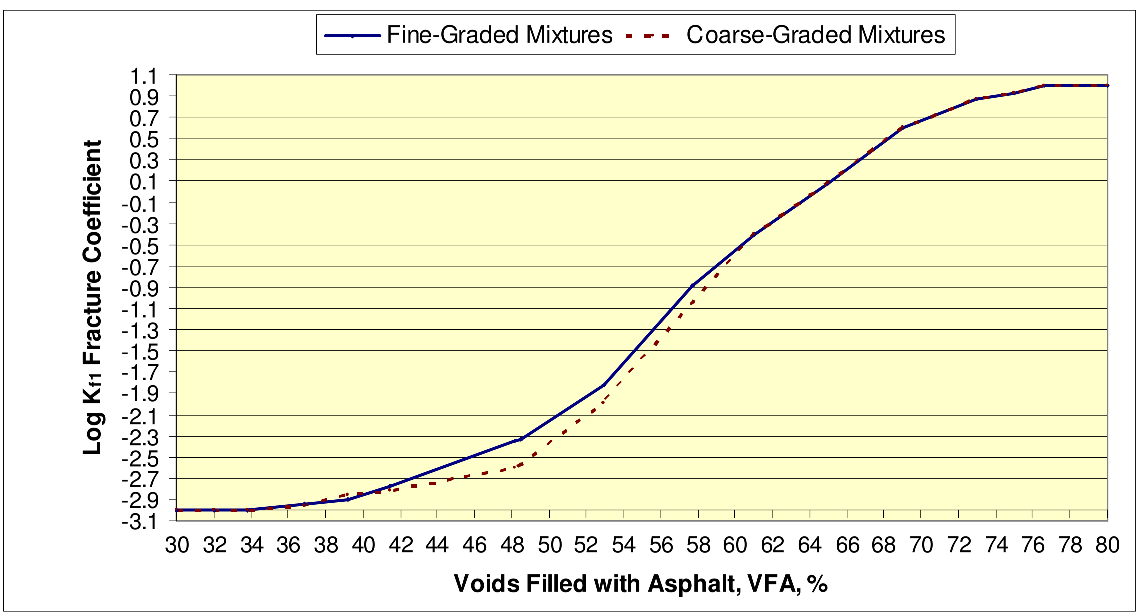


Figure 11-54. Determination of the k_{f1} Parameter from the VFA of the AC Base Mixture.

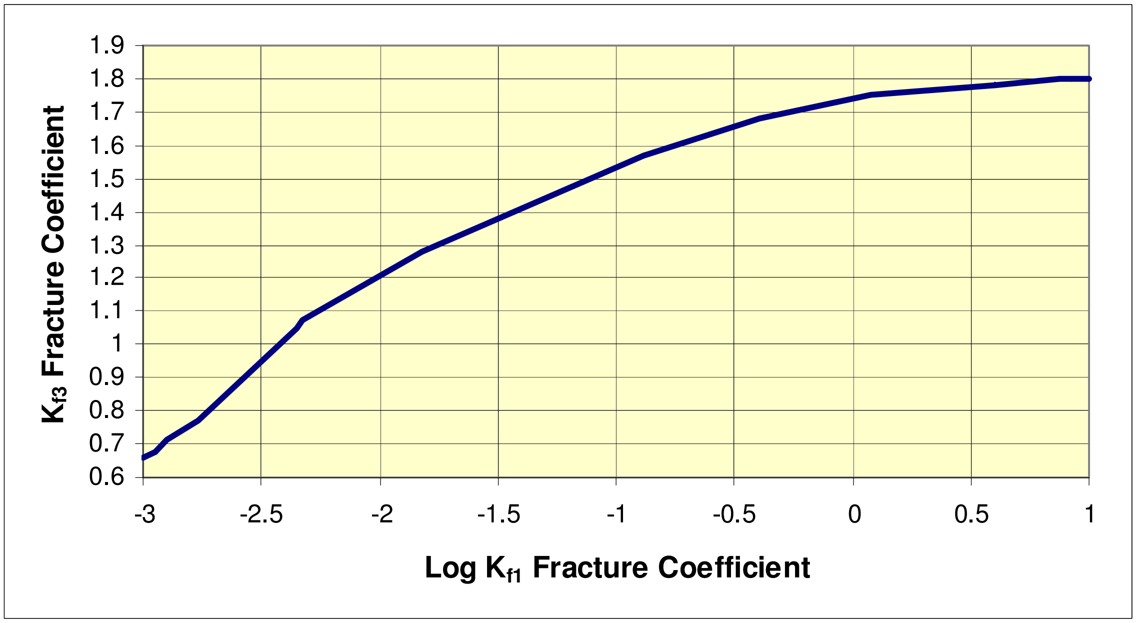


Figure 11-55. Determination of the kf3 Parameter from the kf1 Parameter of the AC Base Mixture.

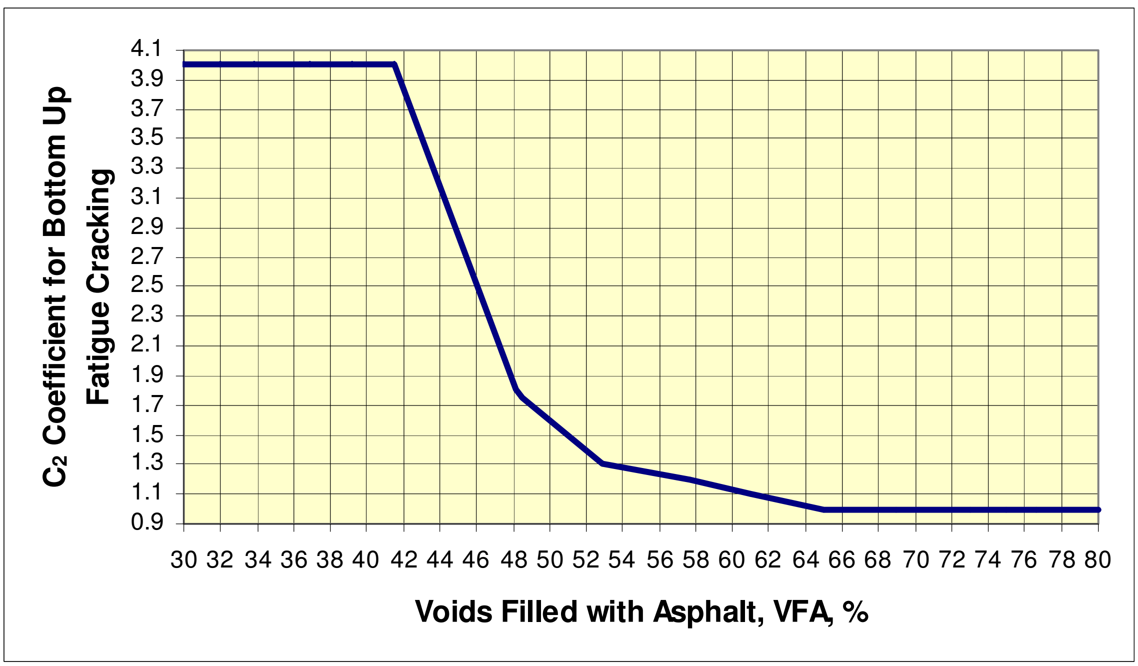


Figure 11-56. Determination of the C_2 Parameter from the VFA of the AC Base Mixture.

AC Base Mixture Ranking based on Performance Predictions

The procedure and equations outlined above were used to estimate the plastic deformation and fatigue fracture coefficients for each AC base mixture included in the study. Total rut depth and bottom-up fatigue cracking were predicted using a constant structure and site condition factors so the only difference in the predictions was the AC base mixtures. Table 11–28 includes some examples of the results.

As tabulated, the major difference in the AC base mixtures is bottom-up fatigue cracking, varying from 10% to over 50%. Surface cracking or longitudinal cracking also varies but the MEPDG Manual of Practice recommends that longitudinal or top-down cracking not be used to make design decisions because the top-down fatigue cracking relationship in the ME Design software will be replaced in the future. Rut depths only varied from 0.23 in (5.8 mm) to 0.44 in (11.2 mm) – all below the recommended threshold value or design criteria. As such, only the bottom-up fatigue cracking predictions were used to rank each of the AC base mixtures.

Three categories of performance or fatigue cracking were used to rank the AC base mixtures, as listed below:

- Crack resistant, defined as less than 25% predicted fatigue cracking over 20 years.
- Average, with 25% to 40% predicted fatigue cracking over 20 years.
- Crack prone, having greater than 40% predicted fatigue cracking over 20 years.

Table 11–28. Predicted Distresses and Performance Indicators for Selected AC Base Mixtures, as examples.

Mix	Project ID	Asphalt PG	Asphalt Content	Air Voids (%)	VMA	VFA	Gradation Index	Predicted Distresses at 20 years					
								Fatigue Cracking (%)	Surface Cracking	Rut Depth (in)	Rut Depth (mm)	IRI (in/mi)	IRI (m/km)
301	MER-219-872	64-22	6.83	6.99	13.83	49.41	8.5	10.0	135	0.30	7.6	133.3	2.079
302	DEL-23-19.24	64-22	8.45	7.01	15.46	54.70	27	21.6	1800	0.34	8.6	140.7	2.195
302	SUM-77-13.54	64-22	8.45	6.98	15.60	55.25	16	30.6	2980	0.23	5.8	140.7	2.195
302	LAK-2-7.60	64-22	8.47	7.00	12.80	54.75	12	34.5	2620	0.30	7.6	145.0	2.262
302	CLA-70-13.98	64-22	8.91	7.00	15.91	56.00	18	35.1	3650	0.44	11.2	151.4	2.362
302	MED-71-5.94	64-22	9.04	7.00	16.04	56.40	20	36.1	3900	0.24	6.1	144.1	2.248
302	LAK-2-7.60	64-22	9.01	7.00	16.70	56.28	7	38.9	3890	0.31	7.9	147.8	2.306
302	MER-219-872	64-22	7.65	7.00	14.64	52.30	8.5	50.6	3980	0.30	7.6	152.3	2.376

These categories of cracking were used to segregate the different AC base mixtures because estimating from the job mix formula (JMF) data does not represent the actual mixture that was placed along the roadway of each project; the JMF provides only an estimate of the volumetric properties. Table 11–29 is a listing of all AC base mixtures and the associated performance rank based on the predicted bottom-up cracking values at 20 years. “NA” was used to identify those AC base mixtures with insufficient mixture design information to complete the adjustment procedure for the bottom-up fatigue cracking coefficients.

Table 11–29. Summary of Performance Rank Predicted using ME Design Software for Bottom-Up Fatigue Cracking.

	Performance		
Project ID Number	Crack Resistant	Average	Crack Prone
FRA/LIC-161-23.20/0.00	A		
LIC-161-1.83		B	
CLI-73-6.52			C
GAL-35-8.32		B	
ROS-207-0.00	NA	NA	NA
WAY-30-11.86 "301"		B	
FAI-33-17.44 SR 13	A		
MER-219-14.04 "301"		B	
FAI-33-13.25	A		
FAI-33-17.44	A		
MAD-142-0.49	NA	NA	NA
VIN-50-11.75		B	
Totals for Mixture 301	4	5	1
FRA-71-5.29	A		
FRA-270-21.67		B	
FRA-270-35.41 Part 2	NA	NA	NA
HAN-30-3.00	NA	NA	NA
LUC-75-6.70		B	
WOO-75-2.37		B	
WOO-75-19.43			C
ATH-33-11.74	A		
CUY-71-0.00	A		
CLA-70-13.98		B	
DEL-23-17.64	A		
LAK-2-7.60		B	
CLI-73-12.03	NA	NA	NA
BUT-75-5.91	A		
DEF-24-7.96			C
FRA-70-3.41		B	
SUM-77-17.20		B	
HAN/WOO-75-19.22/0.00			C
WAY-30-11.86 "302"	A		
FAI-33-19.79	NA	NA	NA
MER-219-14.04 "302"			C
ASD-39-0.00			C
FAI-33-7.31	NA	NA	NA
FAI-33-19.79 SR 18	NA	NA	NA
LUC-2-21.24			C

Project ID Number	Performance		
	Crack Resistant	Average	Crack Prone
WAR-71-14.20			C
FUL-20-10.86	NA	NA	NA
WOO-795-2.01		B	
CUY/SUM-77-0.00/32.73	A		
COS-36-20.83	A		
DEL-23-19.24		B	
Totals for Mixture 302	8	9	7

MED-42-17.68		B	
WOO-75-10.61	A		
MRW-71-3.17	A		
WAR-71-3.78		B	
MED-71-9.56		B	
FAY-35-2.57	NA	NA	NA
GRE/FAY-35-26.20/0.00		B	
HAM-264-6.87	A		
Totals for Unknown Mix	3	4	0

Comparison between Performance Categories and Test Results

Effect of Long-Term Aging

The age of the mixture is important and is difficult to account for in comparing the results from mixtures that have been placed at different times. The ME Design software has an aging model included to harden the asphalt over time consistently between the different projects. The cores, however, used to measure the different AC base properties represent different ages, so aging is a confounding factor between the different AC base mixtures. Figure 11-57 through Figure 11-59 were prepared to illustrate the effect of aging or time between the different projects.

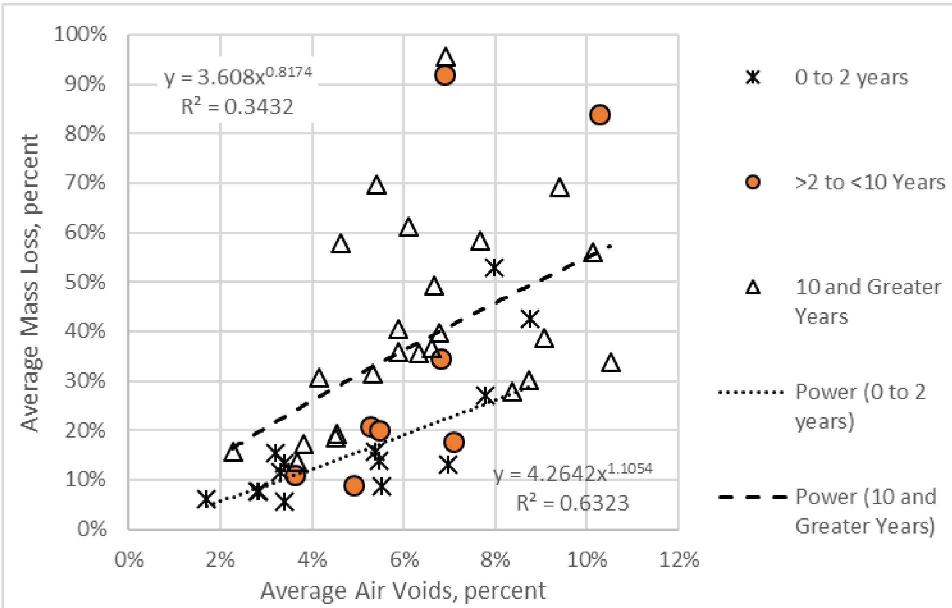


Figure 11-57. Average Mass Loss versus Average Air Voids for different Ages of the AC Base Mixtures.

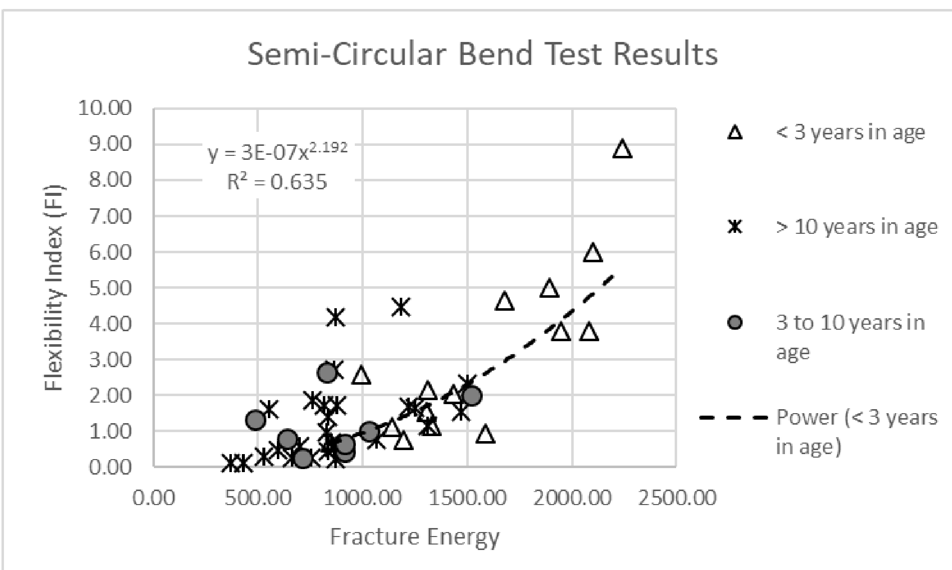


Figure 11-58. Flexibility Index versus Fracture Energy for different Ages of the AC Base Mixtures (Fracture Energy in J/m^2, 1 J/m^2 = 0.000088 BTU/ft^2).

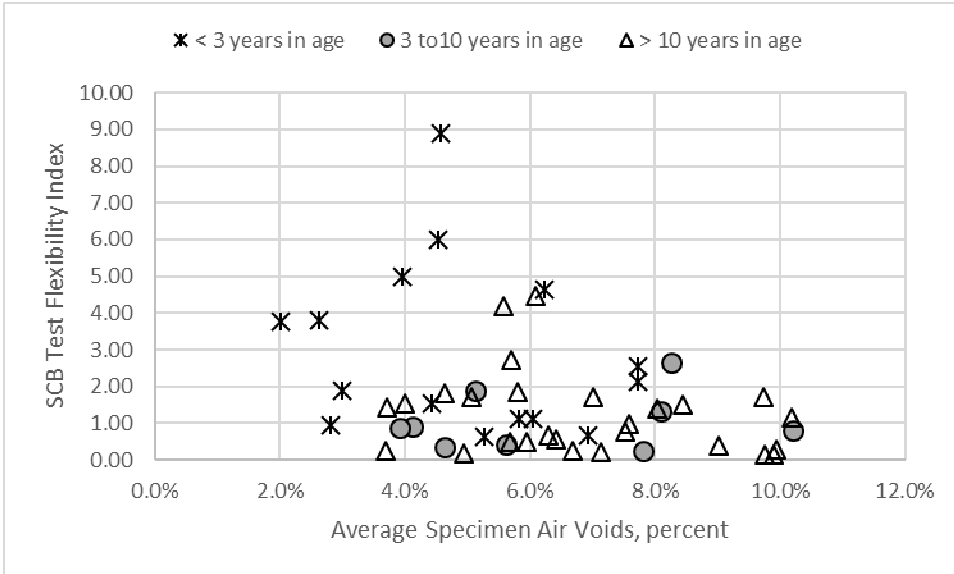


Figure 11-59. Average Flexibility Index versus Average Air Voids for different Ages of the AC Base Mixtures.

- As shown in Figure 11-57, the average mass loss of the AC base mixtures is related to the air voids; the higher the air voids, the greater the mass loss. More importantly, the greater the mass loss over time as shown by those mixtures less than 2 years in age compared to the base mixtures greater than 10 years in age.
- Figure 11-58 compares the flexibility index to the fracture energy from the SCB test. As shown, the flexibility index and the fracture energy become lower over time suggesting a more brittle material.
- Figure 11-59 compares the flexibility index to the average air voids for different AC base mixtures. As shown, the flexibility decreases with the higher air voids and are lower for the older mixtures.

The other important observation is related to the average air voids. Air voids of AC mixtures generally decrease over time because of additional densification from traffic. For the AC base mixtures, however, the change in air voids over time is much less because the base mixtures are much lower in the pavement structure and less susceptible to additional densification due to traffic. Figure 11-60 shows the average air voids versus age for the different AC base mixtures. Other observations made from the laboratory test data are summarized below relative to the performance of mixtures.

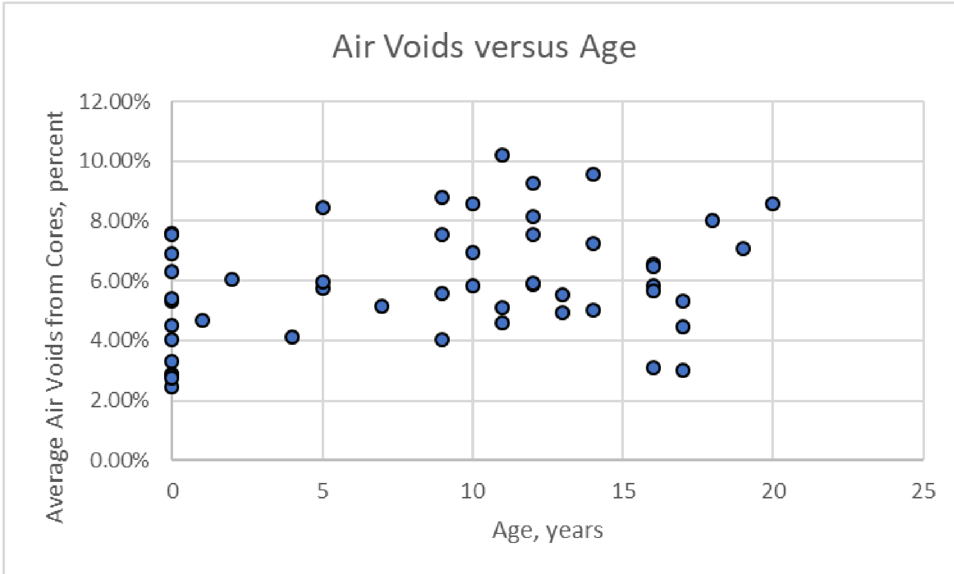


Figure 11-60. Average Air Voids versus AC Base Mixture Age.

- Figure 11-61 shows the change in average mass loss with base mixture age. The general trend is that the mass loss increases with mixture age, but the correlation is poor. In addition, mass loss is more important for the wearing surface mixtures, than for the base mixtures.
- Figure 11-62 and Figure 11-63 show the change in the average flexibility index and fracture energy with base mixture age. There is no significant correlation between the flexibility index and fracture energy with mixture age because the two mixture properties are is more related to the average air voids (see 11.57).
- Figure 11-64 and Figure 11-65 show the change in the average tensile strength ratio (TSR) and dry tensile strength of the base mixture with age. There is no significant correlation between the two TSRs and age and no correlation between the dry tensile strength and age. Figure 11-66 shows the relationship between the dry tensile strength and air voids of the mixture. Indirect tensile strength is usually significantly related to air voids – the higher the air voids, the lower the tensile strength. The reason for the independence is probably related to other mixture properties that vary between the different projects.
- Figure 11-67 shows the relationship between the average dry tensile strength and average flexibility index of the AC base mixtures. As shown, as the flexibility index increases, the dry indirect tensile strength generally decreases. Figure 11-67 also illustrates the importance of mixture age of this comparison.

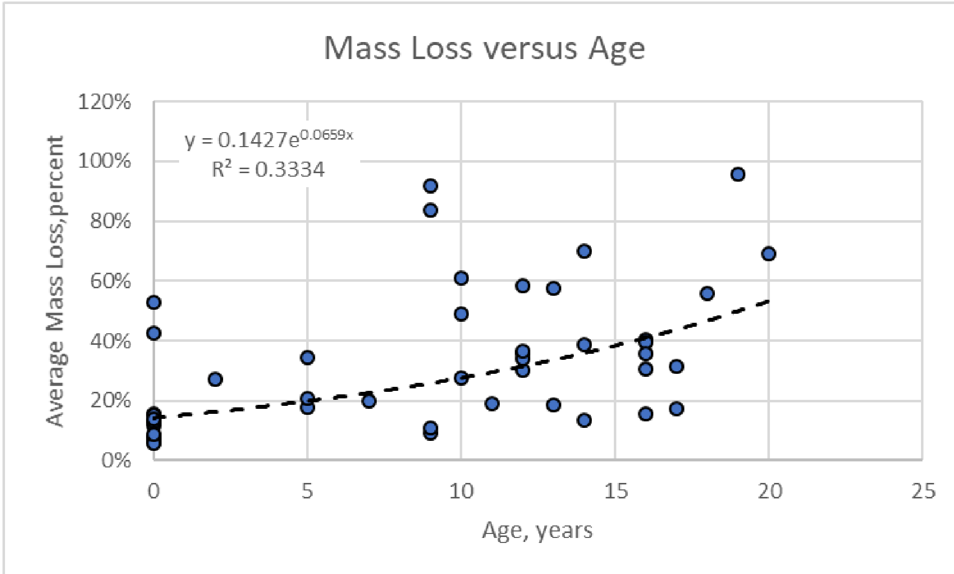


Figure 11-61. Average Mass Loss with Mixture Age.

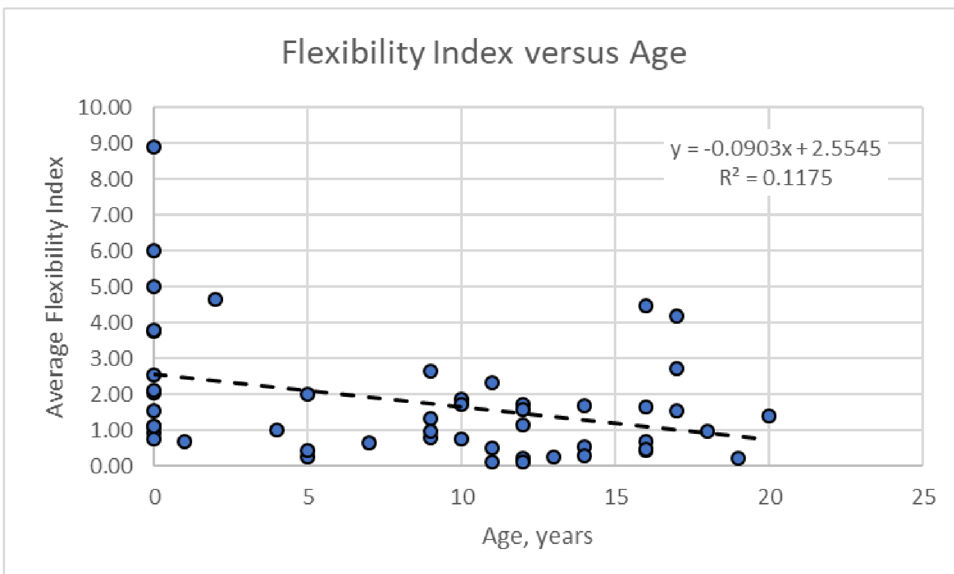


Figure 11-62. Average Flexibility Index with Mixture Age.

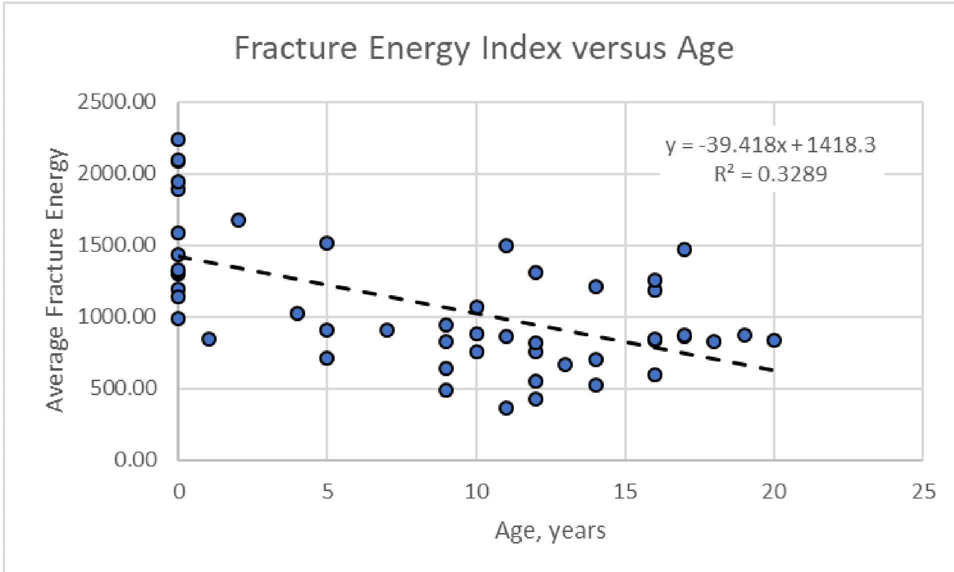


Figure 11-63. Average Fracture Energy with Mixture Age (Fracture Energy in J/m², 1 J/m² = 0.000088 BTU/ft²).

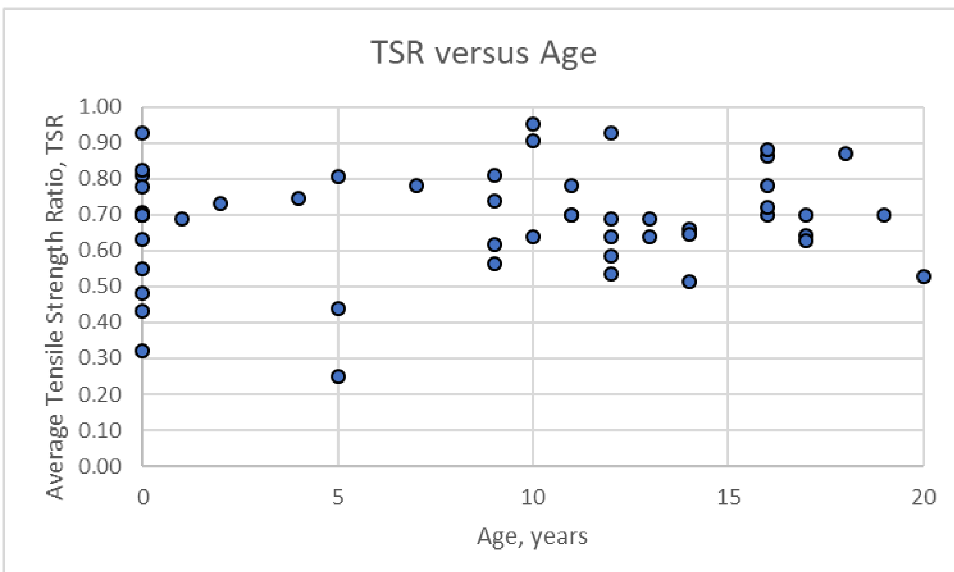


Figure 11-64. Average TSR with Mixture Age.

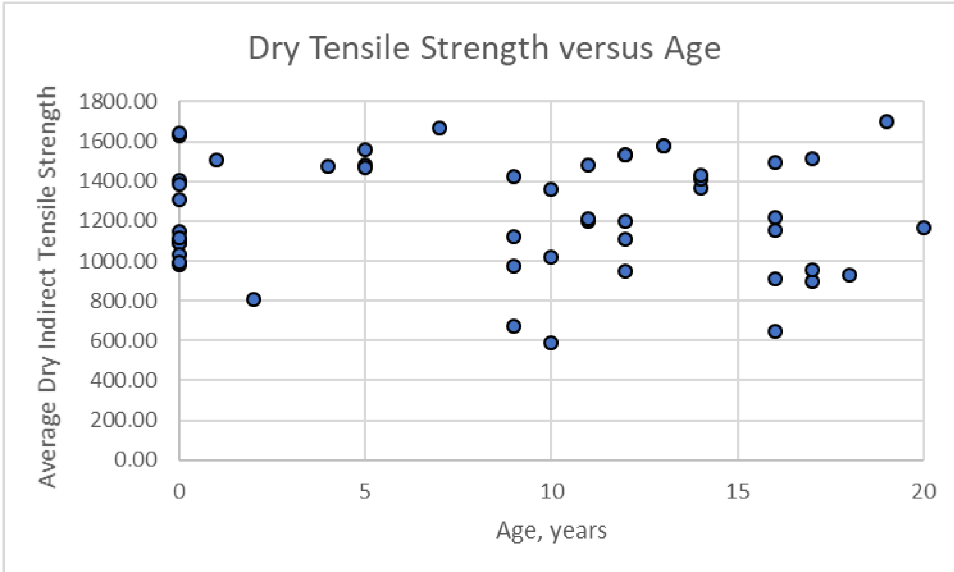


Figure 11-65. Average Dry Tensile Strength (Unconditioned Mixture) with Mixture Age (TSR in psi, 1 psi = 6.89 kPa).

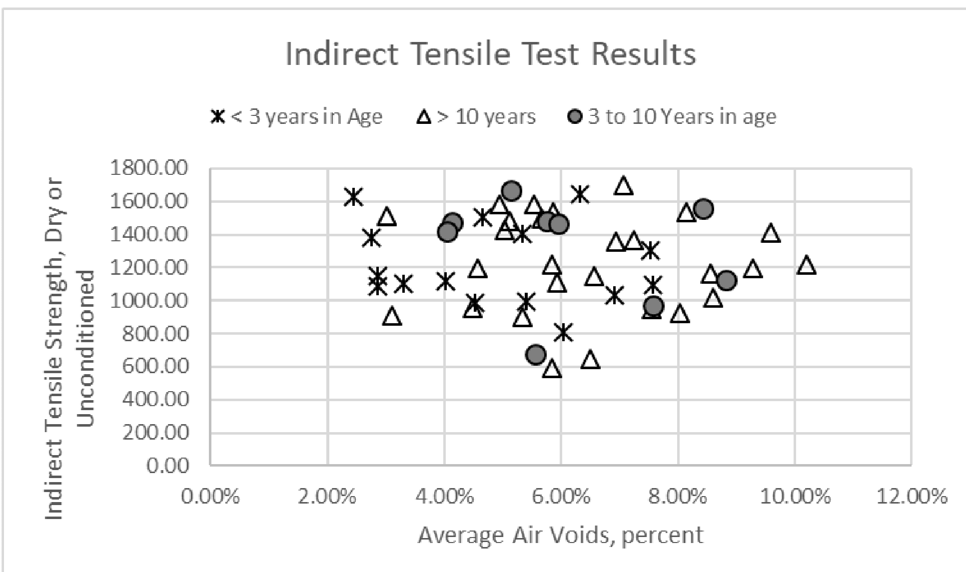


Figure 11-66. Average Dry Tensile Strength (Unconditioned Mixture) with Average Air voids (TSR in psi, 1 psi = 6.89 kPa).

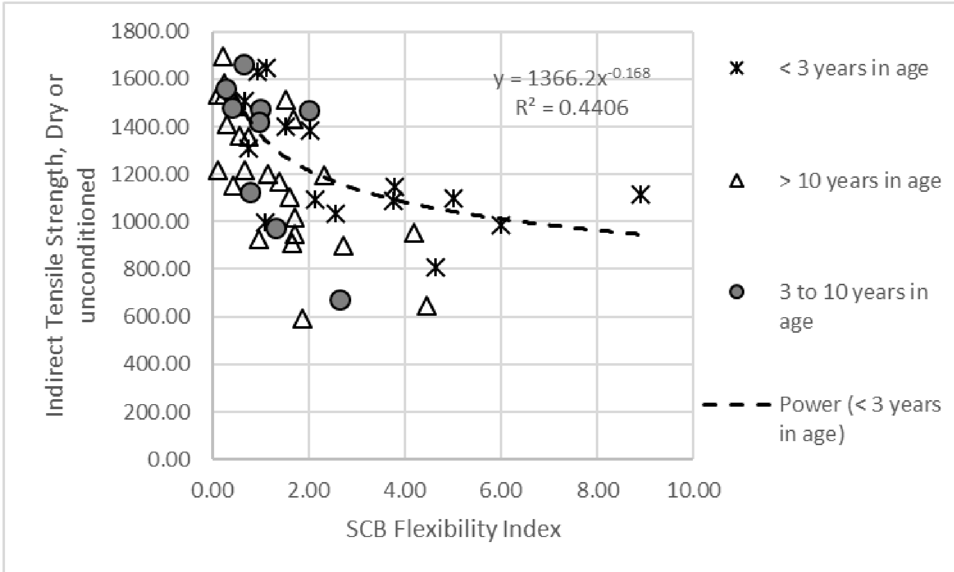


Figure 11-67. Dry Indirect Tensile Strength versus Flexible Index for the AC Base Mixtures (TSR in psi, 1 psi = 6.89 kPa).

An overall comparison was made between the performance prediction categories and the categories of different mixture properties measured in the laboratory of the cores recovered from each project, two of which are noted below.

- Table 11–30 includes a comparison of the fatigue cracking resistant categories and the average flexibility index from the SCB test category. As shown, no correlation was found between the two categories. (NOTE: the cells in Table 11-27 which are not highlighted represent the same mixture observation.) Two reasons for this observation are first, the flexibility index is age dependent and second, the SCB test results are probably heavily dependent on the size of the aggregate and/or gradation of the mixture.
- Table 11–31 includes a comparison of the fatigue cracking resistant categories and the average dry indirect tensile strength category. As shown, there is a reasonable correlation between the fatigue cracking category and indirect tensile strength category. A couple of reasons for this observation is that the dry indirect tensile strength was found to be independent of mixture age and the indirect tensile strength is less dependent on aggregate size. More importantly, gradation is taken into consideration when determining the fatigue cracking fracture coefficients.

Table 11–30. Comparison of the Flexibility Index Category and Fatigue Cracking Category; Number of AC Base Mixtures.

Flexibility Index	Crack Resistant	Average	Crack Prone
>1.5	5	8	5
0.75 to 1.5	4	6	3
<0.75	6	3	0

Table 11–31. Comparison of the Dry Indirect Tensile Strength Category and Fatigue Cracking Category; Number of AC Base Mixtures.

Dry Tensile Strength Category	Crack Resistant	Average	Crack Prone
>1400 psi (9650 kPa)	12	4	0
1000 psi (6890 kPa) to 1400 psi (9650 kPa)	2	11	1
<1000 psi (6890 kPa)	1	3	7

Summary of Comparisons

The following summarizes the results from the mixture performance evaluation as compared to the laboratory test results on cores recovered from each project.

1. Air voids, asphalt content, and gradation are important volumetric properties that have an impact on the fatigue strength of AC mixtures. Air voids and asphalt content are the most important and the design asphalt content is related to the gradation of the mixture. The design air voids and target asphalt content have a significant effect on the fatigue cracking coefficients in accordance with the ME Design software. As such, it is recommended that the air voids (percent compaction) or mix density be used or considered for future work to improve Ohio's AC base mix specification.
2. The TSR value should be considered in approving AC base mixtures, but it did not appear to be related to the performance predictions using the adjusted fracture strength coefficients of the base mixtures. As such, the TSR should be used to determine whether an anti-stripping additive should be added to the mixture.
3. The dry indirect tensile strength was found to be more related to the fracture or fatigue strength coefficients. As such, the dry indirect tensile strength should be considered for use in designing and/or specifying AC base mixtures.
4. There is a lot of variability in the results from the SCB test and the test results were found to be dependent on mixture age. As such, age is a confounding factor and needs to be considered through short and/or long-term aging if the test is to be used for future work and in specifying AC base mixtures.
5. The Cantabro or mass loss is dependent on mixture age. The test was developed more for open-graded wearing surface mixtures, but can be used to segregate the AC base mixtures. No comparison was found between the Cantabro test results and performance categories. As such, age and durability need to be considered through long-term aging if the test is to be used for future work and in specifying AC base mixtures.

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