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UTILIZING LAB TESTS TO PREDICT ASPHALT CONCRETE OVERLAY PERFORMANCE

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ICT PROJECT R27-161
CONSTRUCTION AND PERFORMANCE MONITORING OF VARIOUS ASPHALT MIXES

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16. Abstract

A series of five experimental projects and three demonstration projects were constructed to better understand the performance of pavement overlays using various levels of asphalt binder replacement (ABR) from reclaimed asphalt pavement (RAP), recycled asphalt shingles (RAS), and crushed concrete. The ABR varied from 15% to 60% in the experimental sections. The study of these projects prior to construction, during construction, and for a short monitoring period after construction is intended to determine the impact of various pre-existing pavement conditions, pavement cross-section, mix design, and material properties on the ultimate performance of the asphalt concrete (AC) overlay. This final report is the third report on this research project. Two interim reports that documented project construction and performance to date have been previously published. This report documents final material testing, performance data collection of distress and profile surveys after construction. This report provides a compilation of findings and recommendations from all stages of these projects. The testing suite included Cantabro, stability/flow, Texas overlay, Illinois Flexibility Index Test (I-FIT), fatigue, modulus, creep, and Hamburg rutting. Pavement performance as measured by transverse cracking was found to be more pronounced in thin AC overlays than in thick AC overlays. The Flexibility Index was found to correlate to transverse cracking and confirmed the validity of using this parameter in mix design development.

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EXECUTIVE SUMMARY

Recent actions in Illinois to increase the amounts and types of recycled materials in the production of asphalt concrete (AC) mixes raised questions about the impacts on durability and cracking potential of the subsequent AC pavements being constructed. Policies that allowed 100% of the asphalt binder in reclaimed asphalt pavement (RAP) to replace an equal component of virgin liquid asphalt binder were extended beyond the suggested 20% threshold, and also equally accepted the asphalt binder from a new source, recycled asphalt shingles (RAS). To be truly sustainable, AC mixes with high asphalt-binder replacement (ABR) must perform equivalent to otherwise comparable virgin and low-recycle AC pavements. To better quantify the life-cycle cost and performance of pavement overlays using higher amounts of RAP than had been used until recently, and RAS, a series of five experimental projects were constructed in 2014 and 2015. ABR levels in these pavements varied from a low of 15% to a high of 48%. Additionally, the study included three total-recycle asphalt (TRA) overlays constructed in 2013 as part of a demonstration of very high recycle ABR levels (up to 60%). Although the AC mix information from these 2013 projects is limited, all performance data were able to be collected. This report is the third and final report of the study. Two interim reports were published that presented the construction and initial testing, and the distress and profile surveys conducted before and after construction. This final report includes the results from testing samples of the AC surface and binder courses obtained over a period of three years for basic characteristics, plus Cantabro, stability/flow, Texas overlay, Illinois Flexibility Index Test (I-FIT), fatigue, modulus, creep, and Hamburg Wheel-rutting.

Performance data were collected after each winter resulting in data sets spanning two to four years of pavement life, depending on the project construction date. A few of the sections constructed in 2013 showed increasing amounts of cracking distress. Although the distress is related to the underlying structural conditions, surface AC mixes affect the propagation rate of reflective cracking and the development of cracks due to brittleness. Transverse cracking varied from project to project, but correlated to two main factors, namely pavement overlay thickness family and Flexibility Index (FI) values. Pavements in the "thin" overlay family (3 in or less AC placed directly on concrete pavement) showed more transverse cracking than "thick" family pavements (3.5 to 6 in of pre-existing AC left in place between the concrete pavement and new AC resurfacing). Surface AC mixes with lower FI values, obtained from Illinois Flexibility Index Test (I-FIT) conducted on plant-sampled mix, resulted in more cracking in both pavement families. The Illinois Department of Transportation (IDOT) is currently utilizing a minimum FI value of 8.0 for AC mixes in demonstration projects. Regressions of data in this study indicate that FI values above 8.0 are showing improved performance over AC mixes with lower FI values, for both thick and thin pavements. The FI of AC mixes, sampled at the mix plant, exhibited good inverse correlation to amounts of transverse cracking on the surface AC mixes studied. A reduction of the Condition Rating Survey (CRS) value, the main pavement condition indicator for Illinois, was found to accelerate over time in overlays with lower FI values. Thus, screening out low FI mixes provides the opportunity to increase pavement life.

Although level binder was not a detailed focus of this study, it was noted that the 4.75 level binder used on these projects and the AC surface mixes have similar FI values even though a polymerized PG70-28 was utilized in the level binder mix at relatively high binder content. The use of high ABR in

the level binder mixes hindered their corresponding FI values and appear to increase their cracking susceptibility.

It is recommended that I-FIT, with an FI threshold of 8.0, continue to be adopted as proposed by IDOT. The 4.75 level binder mix needs to have a higher FI threshold than typical AC mixes in order to provide a crack mitigation benefit. With Hamburg Wheel-Tracking specification controls in place, the resulting AC mixes will provide a balanced mix design with reduced risk of both rutting and cracking. The use of secant modulus, as a third performance criterion is recommended; especially since no extra testing is required. The results are readily available from I-FIT data.

TABLE OF CONTENTS

CHAPTER 1: INTRODUCTION

This study follows the field performance of 12 unique AC surface mixes in nine projects from time of construction to document distress development in relation to AC mix properties. The surface mixes utilized asphalt binder replacement (ABR) levels from 15% to 60% by use of reclaimed asphalt pavement (RAP) alone or RAP and recycled asphalt shingles (RAS) together. Plant mix materials were gathered to determine traditional SuperPave® mix properties. In addition, to determine the fundamental engineering properties of each AC mix, the mixes where subjected to a suite of tests, including the recently developed Illinois flexibility index test (I-FIT) for AC (Al-Qadi et al. 2015; Doll et al. 2017a, 2017b; Ozer et al. 2016a, 2016b, 2017). The nine projects include four constructed in 2013, two in 2014 (each has two AC mixes), and three in 2015 (one project has two AC mixes). Projects contained either a single surface mix over the entire project or two mixes on a project, with different mixes by direction. Mixes placed in 2013 and 2015 utilized total-recycle asphalt (TRA), which contains up to 60% asphalt binder replacement (ABR) and 100% recycled aggregates. Distress surveys were conducted prior to construction and each spring, ending in 2017. A reduced suite of tests was performed annually on obtained pavement cores.

Two prior reports under this study documented the construction details of each section— the location, AC mix testing, and early performance (Lippert et al. 2016, 2017). This report summarizes the work effort and presents the final datasets with analysis of the study as a whole.

CHAPTER 2: RESEARCH PROJECT DESCRIPTION

2.1 STUDY GOAL

The goal of this study was to document the testing, construction, and performance of surface AC mixes with a variety with ABR levels, ABR types, and different asphalt binder grades, to allow the evaluation and comparison of the impact of recycled materials on pavement performance.

2.2 STUDY PARAMETERS

The study evaluated a variety of AC mixes with different ABR levels and types (RAS and RAP). Virgin asphalt binder grades were also varied to determine the ability of softer asphalt grades to counter aged asphalt from recycled materials.

The main tasks in this study were as follows:

- Document in detail the pavement condition prior to construction.
- Monitor construction work for cross-sectional or installation issues that may present performance problems later.
- Collect quality assurance information for the record.
- Sample mixes and core pavements for laboratory material characterization over time.
- Monitor pavement performance over time to document performance trends.
- Provide reporting of data available during the study period.

The original project consisted of monitoring five overlay projects that were part of the June 13, 2014, IDOT letting. These projects focused on the surface-mixture components to be placed. Known ABR levels and types of binder replacement (RAP and/or RAS) were the main parameters considered when the experiment was designed. Although all projects in the study were known to have rigid pavement as part of the cross-section, the details of these sections received limited attention during the setup phase of the paving projects.

After approximately one year into the study, three demonstration projects and a comparison project that were let on April 26, 2013, were added to supplement this study. These additional project were constructed to demonstrate use of TRA. Project monitoring on these projects was limited to normal project documentation and material acceptance. The main monitoring up to the time of inclusion in this study was distress surveys and limited mix sampling for a limited suite of tests.

All projects were located in IDOT's Region 1/District 1. Four projects were in Cook County, and five projects were in Will County. Table 2.1 presents the AC surface-mix study matrix including the April 26, 2013, let projects and those from the June 13, 2014, letting. Figure 2.1 presents a general location map of the study sections. The pavements under study were a combination of urban curb-and-gutter, and rural cross sections with open-ditch drainage. The pavement families included bare-concrete pavements; thin existing AC over concrete pavements; thick existing AC over of concrete pavements that were widened; and a short, full-depth AC pavement—all of which needed improvement. Detailed location maps for each project, along with mix designs and cross sections showing existing and proposed improvements, are available in the interim reports (Lippert et al. 2016, 2017).

Table 2.1. Projects and General Study Parameters

April 26, 2013, Letting Projects																
Year	**Project**	**Letting Item[1]**	**Contract**	**Net Length (mi)**	**Surface Mix Details**									**Mix Designs**		**Pavement Family[4]**
					Dir.	**Mix ID**	**Mix**	**ABR %**	**RAS[3] %**	**RAP[3] %**	**AC %**	**Virgin PG**	**Surface Tons**	**Surface**	**Level Binder**	
2013	26th St. (Chicago Heights) from Western Ave to East End Ave	4	60L62	2.0	Both	L62-137M	N50T RA[2]	60	4.5	51	6.7	52-28	3,060	81IT137M	81BIT121M	Thin
2013	Harrison St. (Hillside) from IL 38/Roosevelt Rd. to Wolf Rd.	28	60N67	1.1	Both	N67-338K	N50T RA[2]	56	5.0	53	6.5	52-28	2,131	81BIT338K	81BIT300K	Thin FD HMA
2013	Richards St. (Joliet) from 5th Ave to Manhattan Rd.	31	60P70	0.9	Both	P70-138Z	N50T RA[2]	37	None	27	5.8	58-28	2,223	81BIT138Z	81BIT137Z	Thin
2013	Wolf Rd. (Hillside) from IL 38/Roosevelt Rd. to Harrison St.	9	60M30	0.5	Both	M30-306K	N70 Mix D	20	None	30	5.9	58-28	1,382	81BIT306K	81BIT300K	Thin
June 13, 2014, Letting Projects																
Year	**Project**	**Letting Item[1]**	**Contract**	**Net Length (mi)**	**Surface Mix Details**									**Mix Designs**		**Pavement Family[4]**
					Dir.	**Mix ID**	**Mix**	**ABR %**	**RAS[3] %**	**RAP[3] %**	**AC %**	**Virgin PG**	**Surface Tons**	**Surface**	**Level Binder**	
2014	Crawford Ave./Pulaski Rd. from 172nd to US Rt.	30	60Y03	1.5	S	Y03-157M	N70-30	30	5.0	10	5.7	58-28	2,150	81BIT157M	81BIT147M	Thin
					N	Y03-156M	N70-15	15	2.5	5	5.6	64-22	2,150	81BIT156M		
2014	US 52 from Chicago St. (IL 53) to Laraway Rd.	29	60Y02	3.3	E	Y02-140M	N70-30	30	3.1	20	5.5	58-28	2,320	81BIT140M	81BIT141M	Thin Thick
					W	Y02-159M	N70-30	30	None	34	6.0	58-28	2,320	81BIT159M		
2015	US 52 from Laraway Rd. to Gouger Rd.	16	60N08	3.3	Both	N08-185M	N70 TRA[2]	48	5.0	39	6.0	52-34	5,236	81BIT185M	81BIT163M	Thick
2015	US 52 from Gouger Rd. to Second St.	15	60N07	1.5	Both	N07-185M	N70 TRA[2]	48	5.0	39	6.3	52-28	3,014	81BIT185M	81BIT163M	Thick
2015	Washington St. from Briggs St. to US 30 (Lincoln Highway)	31	60Y04	1.9	W	Y04-177M	N70-30	30	3.1	20	6.6	58-34	1,580	81BIT177M	81BIT163M	Thin Thick
					E	Y04-159M	N70-30	30	None	34	6.0	58-34	1,580	81BIT159M		

[1]April 26, 2013, or June 13, 2014, letting item number.
[2]Total-recycle asphalt (TRA: 100% recycled aggregate with high ABR).
[3]Value indicates percentage of mixture of RAP and RAS that contributes to the indicated ABR percentage.
[4]Thin = Thin AC overlay of rigid pavement, Thick = Thick AC overlay of rigid pavement, and FD AC = Full-depth AC
Note: Maximum percentage of RAS allowed is 5% of the total mix, by specification.

2.3 PRE-EXISTING CONDITIONS AND PROPOSED IMPROVEMENTS

In total, nine pavement projects were included in this study. All pavements were in need of rehabilitation, with bare-PCC of the pavement cross sections, conditions, and improvements undertaken for the study pavements. Detailed discussion of cross sections and pavement conditions were provided in a previous report (Lippert et al. 2016, 2017).

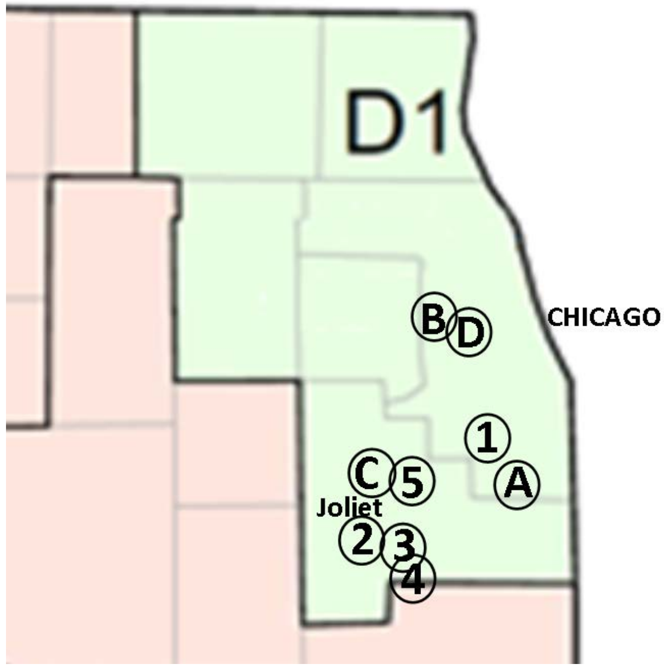


Figure 2.1. Study project approximate locations.

Prior to overlay, the pavement families, their designations based upon the original cross sections and resulting improvements, included bare Portland cement concrete (PCC) pavement; composite pavements; and a short section of full-depth AC pavement. The groupings were composite pavement with “thin” pre-existing AC overlays of 2.5 to 3 in of AC over a PCC pavement, composite pavement with “thick” pre-existing overlays of 5.75 to 8.0 in of AC over a PCC pavement, and short segments of full-depth AC pavement. The full-depth AC pavement data were separated, so as not to comingle results with those of other families, but were insufficient for meaningful analysis as a pavement family itself in this study. Pavement families for each project are included in Appendix A.

All pavement overlays placed in this study ranged from 2.25 to 3.00 in. If the existing pavement was overlaid with AC at 3.00 in or less, the pre-existing overlay was milled off down to bare-PCC pavement in what is commonly called a “mill-and-fill” operation, in which the pre-existing AC overlay thickness removed matched the new overlay thickness being placed. For existing thick-pavement families, the milling removed only the upper pavement layers to the thickness of the new overlay.

Appendix A provides a summary table of the basic cross-section information on pre-existing and proposed pavements, cracking extent, amount of patching, ride, and rutting at rehabilitation for each section. When pre-existing AC overlays were placed on bare-PCC pavement with curb-and-gutter, the plans called for milling to a depth of 1.5 in at the curb line, which tapered to zero at the center of the lane. Level binder was placed on the interior lanes and ended at the start of the taper. The taper area did not receive level binder. This detail is also discussed in the interim reports (Lippert et al. 2016, 2017).

2.4 STUDY APPROACH

Laboratory and field work on the study was split between the Illinois Department of Transportation (IDOT) Bureau of Materials and Physical Research (BMPR) and the Illinois Center for Transportation (ICT), based upon each organization's strengths and capabilities.

2.4.1 Material Sampling and Testing

Due to the timing of project inclusion in the study, there were sampling differences between the 2013 and 2014 let projects. For the 2013 let projects, IDOT's Region 1/District 1 staff sampled plant-produced AC samples that were provided to ICT along with basic production volumetric information and mix designs. ICT conducted Hamburg wheel-tracking and the Illinois flexibility index test (I-FIT) (Al-Qadi et al. 2015; Doll et al. 2017a, 2017b; Ozer et al. 2016a, 2016b, 2017). Material samples of the AC were not obtained for the comparison project on Wolf Road. Table 2.2 summarizes the testing program and responsibility by project letting date.

Table 2.2. Summary of Testing

Test	Specification	Field Core		Plant-Mixed Lab-Compacted		Laboratory
		2013 Let	2014 Let	2013 Let	2014 Let	
Asphalt-binder content	AASHTO T 164-13 (Illinois Modified)			X	X	BMPR
Aggregate gradation	AASHTO T-27 (Illinois Modified)			X	X	BMPR
G_{mm}	AASHTO T 209-12 (Illinois Modified)			X	X	BMPR
Marshall Stability/Flow	ASTM D6927(Illinois Modified)				X	BMPR
Cantabro loss	TxDOT Test-245-F				X	BMPR
TSR	AASHTO T 283-07 (Illinois Modified)				X	BMPR
Texas Overlay Test	Tex-248-F				X	BMPR
Complex Modulus	AASHTO T 342-11			X	X	ICT
Flow Number	AASHTO TP 79-13			X	X	ICT
Beam Fatigue	AASHTO T-321-14				X	ICT
IDT Creep Compliance	AASHTO T 322-07 (2011)	X	X		X	ICT
Hamburg Wheel	AASHTO T 324-11 (Illinois Modified)	X	X	X	X	ICT
I-FIT	AASHTO TP 124 (ITP 405)	X	X	X	X	ICT
Performance-graded asphalt binder	AASHTO M 320	Binder sample from plant				BMPR

Note: Illinois Modified Test procedures may be found in the Manual of Test Procedures for Materials (IDOT 2015)

For the 2014 let projects, the sampling and testing regimes were more structured and followed a detailed study plan. For this group of projects, AC-plant samples were taken jointly by IDOT and ICT. This process consisted of the contractor loading approximately four tons of production AC mix from the storage silo into an end loader and placing it on the ground in a pile. The pile was then stuck off to a thickness of 16 in so that the mix would not shift and possibly segregate during sampling. Samples bags were filled by taking shovel samples around the pile at different locations for each bag. Feedstock materials were also sampled, including aggregates and liquid asphalt binder used for each mix. These materials were obtained so that the mix could be recreated in the lab if the need arose.

After construction of the 2014 let projects, a series of field cores was obtained to represent the "as-built" pavement and then annually. These cores underwent a reduced suite of tests to characterize the changes in material properties with time. Field cores were also obtained for the 2013 let projects, starting in 2015.

2.4.2 Field Data Collection

In a similar manner, the field data collection was split between IDOT and ICT for this project. During construction of the 2014 let projects, ICT team members observed paving operations, in addition to performing the usual job-control checks and documentation. Project documentation was obtained from IDOT for key activities such as patching and sequence of AC laydown. In addition to project monitoring, detailed distress surveys were conducted by IDOT, as well as initial automated video and profile data collection prior to construction. After construction, ICT arranged for automated profile data collection; and IDOT Region 1/District 1 obtained core samples. Table 2.3 summarizes the activities of the field data collection efforts.

Table 2.3. Field Data Collection Efforts

Activity	Data Collection Run		Lead Agency
	2013 Let Projects	2014 Let Projects	
Detailed Crack Survey	Pre-construction[1], Post-construction, 2014, 2015, 2016, 2017	Pre-construction, Post-construction, 2015, 2016, 2017	IDOT
IDOT Profile Survey	Inventory PMS Summary	Pre-construction	IDOT
Road-Profile	2015, 2016, 2017	2014[2], 2015, 2016, 2017	ICT
Core Sampling	2015, 2016, 2017	2014, 2015, 2016, 2017	IDOT R1/D1

[1] Wolf Road pre-construction survey conducted from Google images
[2] For 2014 let projects, a data run was conducted the first winter while pavement was frozen.

2.4.2.1 Crack Surveys

The Illinois Department of Transportation (IDOT) routinely conducts pavement surveys for documenting pavement performance of various experiments. This procedure consists of walking the sections while mapping distresses and severities on survey forms by station. Prior to construction, each project under study received a crack survey to document the condition of the pavement. Cracks were mapped and coded as outlined in IDOT's Pavement Distress Manual (IDOT 2012a). For both the 2013 and 2014 let projects, IDOT conducted surveys prior to improvement, post-improvement, and annually for all sections except Wolf Road. Wolf Road was completed prior to its being identified as a comparison section for the 2013 TRA projects. To obtain pre-construction pavement conditions, Google Street-View images were reviewed and survey sheets developed to meet this need. After construction, surveys were conducted and then annually shortly after the spring thaw.

2.4.2.2 Road-Profile Data

In a similar manner as the crack surveys, road-profile data using automated profile data collection equipment was conducted at key times for the projects.

For the 2013 let projects, the data collection was established with this study effort, thus there are no detailed profile data prior to 2015 on this group of projects. The construction of the April 26, 2013, let projects was originally intended to be a demonstration rather than a detailed research study; as such, pre- and post-construction profile data were not collected. For the pre-construction profile data reported, pavement inventory data just prior to the improvement was utilized. These data were collected by IDOT's vendor in the direction of inventory (normally eastbound or northbound, depending upon route designation). Data in the opposing direction were not collected.

The 2013 let projects were formally added into the study in 2015; and in November of that year, profile data were collected by Engineering & Research International, Inc. (ERI, Inc.) The profile dataset provided averages every 0.01 mi in each direction and lane of the projects. Each data run was reviewed in detail, with editing of tapers, intersections, and railroad tracks, as well as low-speed data (below 15 mph) for removal from the averages. This removal was done due to volatility in the data at these locations from starting/stopping and transitions from old to new pavement, which introduced questionable values into the dataset. After screening, the dataset consisted of values collected at fairly uniform traffic speeds and free of influences of intersections, railroad crossings, and end-section transitions.

For the 2014 let projects (constructed in 2014 and 2015), the pre-construction profile data collection was conducted by IDOT's video-survey vendor. The data were collected in each lane and direction prior to construction. Values of the International Roughness Index (IRI) and rutting were determined every 0.1 mi, as well as averages for the study segments. After construction, initial road profiles on the improved pavement were obtained by ERI, Inc., for the remaining study effort. The data were collected at pre- and post-construction, at frozen-subgrade conditions the first winter after construction, and then annually in spring/summer. The dataset provided averages every 0.01 mi in each direction and lane of the projects. Each data run was reviewed and edited to result in uniform segments, as described above.

2.4.2.3 Construction Monitoring

As noted, the 2013 let projects were originally intended to be demonstration projects for showing the feasibility of utilizing total-recycle asphalt (TRA), which allowed up to 60% asphalt binder replacement (ABR). As such, these projects were not monitored during construction, other than the normal project control and quality assurance for acceptance of the work. Limited plant AC material sampling was conducted and provided to ICT for study. General project information and early performance for the 2013 let projects has been reported elsewhere (Lippert et al. 2014).

Projects let in 2014 received more detailed monitoring of the construction process to document any links of construction activities to long-term performance. During construction, a member of the study team observed key phases of each project. Those phases were milling, patching, priming, and paving. Detailed construction monitoring was reported in two interim reports (Lippert et al. 2016, 2017).

2.4.2.4 Core Sampling

Coring was conducted by IDOT Region 1/District 1 and delivered to the study team at ICT. Due to the 2013 let projects' being added to this study once underway, cores were not obtained until 2015, about two years after construction. For projects let in 2014, cores were obtained after construction and then annually for the duration of the study, to monitor changes in the AC over time.

CHAPTER 3: PRE-EXISTING CONDITIONS AND PROPOSED IMPROVEMENTS

This chapter provides a summary of pre-existing cross sections and proposed improvements. Detailed location maps and breakouts of segments are provided elsewhere (Lippert et al. 2014, 2016, 2017). Speed limits varied from 30 to 55 mph on the study sections. Two-way average daily traffic (ADT) volumes ranged from 1,700 for a local service road to 22,400 for a major arterial. Traffic for each project is presented in Appendix A.

3.1 PAVEMENT SECTIONS

A detailed summary of pre-existing conditions for each project is provided in Appendix A. This summary includes information on the original pavement, proposed pavement-improvement work, and assigned pavement family. Projects were broken into separate study segments if the original pavement cross-section or the number of lanes changed.

The pavements included in this study are typical of Illinois pavements in need of rehabilitation in urban curb-and-gutter sections and rural sections with ditch drainage. With the exception of Harrison Street (contact 60N67), which contains a short section of full-depth AC, all the sections contained jointed-PCC pavement, an AC overlay of jointed-PCC pavement, or a cement-treated, PCC base underlying the existing AC surface, as was the case with Washington Street (60Y04). All sections were in need of resurfacing and part of the district's normal improvement program.

Leading up to the time of rehabilitation, the pavements were subject to various maintenance techniques in order to keep them operating. Maintenance activities included pothole filling, full-depth patching, and intermittent full-width mill-and-fill inlays of the existing AC surface.

PCC pavement thickness ranged from 9 to 10 in, with the rigid base on Washington Street being 8 in. The existing AC overlays that were in place ranged from 2.5 to 8.25 in. Appendix A provides a tabular presentation of the original pavement cross sections by segment.

The improvements for the projects in this study were driven by policies contained in IDOT's *Bureau of Design and Environment Manual,* Chapter 53 on pavement rehabilitation (IDOT 2010). Designers are given limited latitude in thickness selection. The basic rehabilitation is to repair the existing pavement where needed with full-depth patches. For sound jointed-PCC pavements Class B (doweled-concrete) patches are recommended. For older pavements and those in a more deteriorated condition, Class D (full-depth AC) patches are recommended. All projects in this study utilized Class D (full-depth AC) patches. If existing AC is present, the existing AC is to be milled prior to patching. In the case of these projects, the milling thickness ranged from 2.25 to 3.0 in. For the thin AC overlays present at the time of rehabilitation, the milling removed the existing AC down to the PCC pavement. The new overlay then replaced the thickness in kind. Milling down to PCC was done so as not to leave a thin layer of questionable AC between the new overlay and the PCC pavement. Such layers left in place have been known to be a source of rutting on some projects.

The new AC overlay ranged from 2.25 to 3.0 in and consisted of an AC level binder of ¾- to 1-in thick, topped with 1.5 to 2.0 in of AC surface. Except for bare-PCC sections, the surface elevation of the pavement did not change. To maintain drainage function in urban cross sections with curb-and-gutter, bare-PCC pavement sections were milled, starting at the pavement/gutter joint, to a thickness of 1.5 in, tapering to zero 6 ft away. A level binder was not applied in the milled taper area but was present in the remaining lane areas. See previously reported cross sections for these details by project and segment (Lippert et al. 2016, 2017). Appendix A provides a tabular presentation of the pavement-improvement work conducted on each pavement cross-section, by segment.

The AC overlays ranged from 2.25 to 3.0 in, placed on bare-PCC pavement or part of a mill-and-fill operation that left 3.5 to 6 in of AC in place. After completion of the rehabilitation work, one group of pavements resulted in new AC overlays from 2.25 to 3.0 in and was termed a “thin” AC overlay family. Another group of pavements contained a layer of pre-existing AC that remained in place and new AC overlay totaling in the range of 5.75 to 8.25 in and was termed a “thick” AC overlay family. These terms are used throughout this report, as reflected in Appendix A. A third pavement family, is included in this study, namely a full-depth AC section, which is limited to a short segment of the Harrison Street project (contract 60N67).

3.2 PRE-CONSTRUCTION PAVEMENT CONDITION

3.2.1 Distress Surveys

Prior to construction, all the projects received a detailed distress survey by IDOT staff walking and mapping the pavement distress type and severity, as outlined in the *Bureau of Materials and Physical Research Pavement Distress Manual* (IDOT 2012a). Survey sheets were provided to the study team, and the data were summarized. The resulting detailed data are presented in Appendix B. These data were further summarized and included in Appendix A as pre-construction transverse joints and cracks (all distress levels), normalized to ft/1,000 ln-ft and patching as a percent of the pavement area.

3.2.2 Pavement-Profile

Data on ride quality and rutting were collected using high-speed, automated profile data collection. Due to the timing of project construction in relation to this study, road-profile data were either not collected, collected by IDOT’s vendor, or collected under this study.

A review of pavement inventory data, just prior to the improvement, provided pre-construction data for the April 26, 2013 let projects, which originally intended to be for demonstration. These data were collected by IDOT’s vendor every other year in the direction of inventory (normally eastbound or northbound, depending upon route designation) and are an average of the segment. Data in the opposing direction were not collected. The projects let in 2013 were added into this study, with the first data collection by the team conducted in November of 2015 by ERI, Inc. The data were averaged every 0.01 mi in each direction and lane of the projects. Each data run was reviewed in detail with attention given to tapers and intersection data, as well as low-speed data (below 15 mph), all of which were removed from the dataset, as explained above. The resulting data are presented in Appendix C.

For the 2014 let projects, the pre-construction profile data collection was conducted by IDOT's video-survey vendor. The data were collected in each lane and direction prior to construction. Values of the IRI and rutting were determined every 0.1 mi, and averages were determined for the study segments.

After construction of these projects, ERI, Inc., collected the profile data, which were screened as noted above. The resulting data are presented in Appendix C.

CHAPTER 4: CONSTRUCTION MONITORING

As noted, construction monitoring of this study was limited to the 2014 let projects. The study team was informed of contractor schedules and activities by IDOT Region 1/District 1 field staff for each project. The availability of such information allowed team members to concentrate visits and general data collection on critical activities that would impact pavement performance. These activities were milling, patching, tack coating, AC level binder placement, and AC surface placement. These activities were reported in detail previously (Lippert et al. 2016, 2017) and are summarized below for the study as a whole.

Unique for this group of projects is that all the 2014 let projects were constructed by D Construction, Inc., of Coal City, Illinois, the successful bidder and prime contractor for the projects. The result is that construction operations and equipment were essentially the same, as construction moved from project to project in 2014 and 2015.

4.1 PAVEMENT COLD-MILLING

In general, milling per Article 440 and 1101.16 of the *Standard Specifications for Road and Bridge Construction* (IDOT 2012b) was followed. However, Article 440.04states:

> "The temperature at which the work is performed, the nature and condition of the equipment, and the manner of performing the work shall be such that the milled surface is not torn, gouged, shoved or otherwise damaged by the milling operation."

The article also states:

> "When tested with a 16 ft (5 m) straightedge, the milled surface shall have no surface variations in excess of 3/16 in. (5 mm)."

The longitudinal profile requirement of the specifications was generally met for all the projects. However, for both projects constructed in 2014 [Crawford/Pulaski (60Y03) and US 52 (60Y02)], the same milling machine was used and resulted in deep "grooves" over the length of the pavement, where new teeth had replaced lost or damaged teeth in an otherwise worn milling head. Straightedge testing longitudinally with the 16-ft straightedge is intended to address possible ride quality issues from waves and dips that are difficult to see.

The longitudinal "grooving" could be viewed as a longitudinal "gouge" or "otherwise damaged," and the contractor could be instructed to correct the work before proceeding. Though small, deep grooves from milling can impact the yield of the level binder to the point that material may need to be added to the project or the thin level binder layer must be laid thinner than the plans show.

For Crawford/Pulaski (60Y03), where the pre-existing AC overlay was milled down to bare-PCC, highly distressed joints were exposed in both longitudinal and transverse directions. The resulting pavement surface is shown in Figure 4.1a. The most severe areas were patched full-depth using AC; however, a number of these areas were cleaned of loose material and filled as cracks, joints, and flangeways.

For projects constructed in 2015 [US 52 (60N07 and 60N08) and Washington (60Y04)], excessive grooving from the milling head was not a problem. The milling was performed in a uniform manner producing the desired milled texture. What was seen as a problem in the 2015 construction projects was the depth of milling resulting in very thin layers of an AC lift remaining in place above a lift interface. Where the remaining AC layer thickness was below approximately 0.75 in, traffic tended to cause the layer to be lost. This problem was most evident on the Washington Street (60Y04) project. Uniformity of mill texture and loss of AC layer across the pavement lane are shown in Figure 4.1b.

(a) Crawford/Pulaski Road after milling (2014) **(b) Washington Street after milling (2015)**

Figure 4.1. Examples of pavement milling.

4.2 PATCHING, FILLING OF CRACKS, JOINTS, AND FLANGEWAYS

During plan development, an estimate of patching quantities, class, and size is made. For the projects in this study, Class D AC patches were specified. Also detailed were quantities of AC mixture needed for filling cracks, joints, and flangeways. These estimates may be difficult due to existing AC overlays masking pavement issues and/or severe joint and crack distress. The process in the field is to mill the pavement to allow the removal of the distressed surface that may be masking underlying pavement problems, and then to apply the plan quantity of patching to determine if it is adequate or if some adjustments in quantity are needed.

Patching quantity determination is problematic, in that quantities may change from the time of plan development to construction, which leads to quantity and cost overruns. Although some plans may include a patching schedule, the plans on these projects simply contained quantities of Class D patches for different types (sizes). A great deal of engineering judgement is used in plan development for quantities and then, in the field, on where to utilize the plan quantities.

Patching quantities in the plans were sufficient to address needs on all the projects. Once milling was complete and patching locations selected, the plan quantities were not fully utilized on any of the projects. The US 52 projects seemed to be adequately patched; however, the Crawford/Pulaski and Washington street projects may have benefited from utilizing more of the plan quantity. In lieu of patching, distressed areas were cleaned to remove loose material and filled with AC under the provision for filling cracks, joints, and flangeways. Table 4.1 presents a summary of plan and actual quantities by square yard of patching and percent of pavement area patched for each project. Table 4.2 presents a summary of the plan and actual quantities of AC material for filling of cracks, joints, and flangeways. To compare sections, tons of AC used for crack, joints, and flangeways used on a station lane unit (100 ln-ft) for each project were also determined and presented.

Table 4.1. Plan Quantity vs. Actual Quantity Used—Patching

Project		Plan Quantity		Quantity Used		
Description	**Contract Number**	**Patching, SY**	**Patching, %**	**Patching, SY**	**Patching, %**	**Percent of Plan Quantity**
Crawford Ave./Pulaski Rd. from 172nd to US Rt. 6	60Y03	1,515	3.2	891.92	1.9	59
US 52 from Chicago St. (IL 53) to Laraway Rd.	60Y02	510	0.9	387.78	0.7	76
US 52 from Laraway Rd. to Gougar Rd.	60N08	370	0.6	202.2	0.3	55
US 52 from Gougar Rd. to Second St.	60N07	340	1.0	328.7	0.9	97
Washington St. from Briggs St. to US 30 (Lincoln Highway)	60Y04	130	0.4	0.00	0.0	0.0

Note: Patching quantity is the total of all types (sizes) of Class D patching, i.e., pavement removal with AC replacement.

4.3 TACK COAT ON EXISTING PAVEMENT

After repairs were complete, the pavement was cleaned and then tack-coated. The study team was not present during these operation but did observe each roadway before paving and discussed the operation with field staff. The tack coat was applied early in the morning prior to paving so that by the time of paving, traffic had spread tack material to a uniform application (which was observed by the study team). Figure 4.2 shows the resulting uniform tack coat for Washington Street (60Y04).

Weather impacted US 52 (Laraway Road to Gougar Road—60N08) work, with a light rain occurring after application of the tack coat. The rain was sufficient to cancel paving planned for the day. Some slight loss of the tack material did occur but not to the extent that reapplication was required. Figure 4.2 shows an example of the unbroken asphalt emulsion. By midmorning, the light rain dissipated; and the section was drying out, allowing the asphalt material to break.

Table 4.2. Plan Quantity vs. Actual Quantity Used—Filling of Cracks, Joints, and Flangeways

Project		Plan Quantity		Quantity Used	
Description	Contract Number	AC, tons	Tons per 100 ln-ft	AC, tons	Tons per 100 ln-ft
Crawford Ave./Pulaski Rd. from 172nd to US Rt. 6	60Y03	100	0.34	73.06	0.25
US 52 from Chicago St. (IL 53) to Laraway Rd.	60Y02	84	0.31	84.41	0.31
US 52 from Laraway Rd. to Gougar Rd.	60N08	91	0.27	20	0.06
US 52 from Gougar Rd. to Second St.	60N07	54	0.35	21.85	0.14
Washington St. from Briggs St. to US 30 (Lincoln Highway)	60Y04	48	0.34	9.81	0.07

Figure 4.2. Brown, "unbroken," emulsified-asphalt prime coat on US 52 Laraway Road to Gougar Road—contract 60N08.

It should be noted here that in the years prior to these projects, IDOT undertook an effort to increase the application rate of tack coats and improve the uniformity of application. The uniformity of the end product prior to paving on the study projects was sufficient and met the intention of the specifications (Al-Qadi et al. 2012).

4.4 LEVEL BINDER

All projects in the study utilized an IL-4.75 (sand-mix) level binder that is typically specified in IDOT Region 1/District 1. This mix used a PG 70-28 asphalt binder. Recycled materials (RAP and RAS) were allowed in the level binder, with an asphalt binder replacement (ABR) of 29 to 35%

for the projects. The ABR split between RAP and RAS was approximately 50/50. Mix design and cross-section details were previously reported (Lippert et al, 2015, 2016). The IL-4.75 level binder mixes used on the study projects were similar to level binder mixes used throughout IDOT Region 1/District 1 in 2014 and 2015.

Placement of the level binder was either 0.75 or 1.0 in thick, depending on the project. Unique to the level binder placement was the width of placement and the milling treatment of bare-PCC pavement cross sections. To preserve curb-and-gutter height for drainage in urban sections, the PCC pavement was milled 1.5 in in depth to accommodate the surface thickness at the gutter/pavement edge. The milling thickness was tapered to zero mid-lane, 6 ft away from the joint, and where the level binder would start, as shown in Figure 4.3.

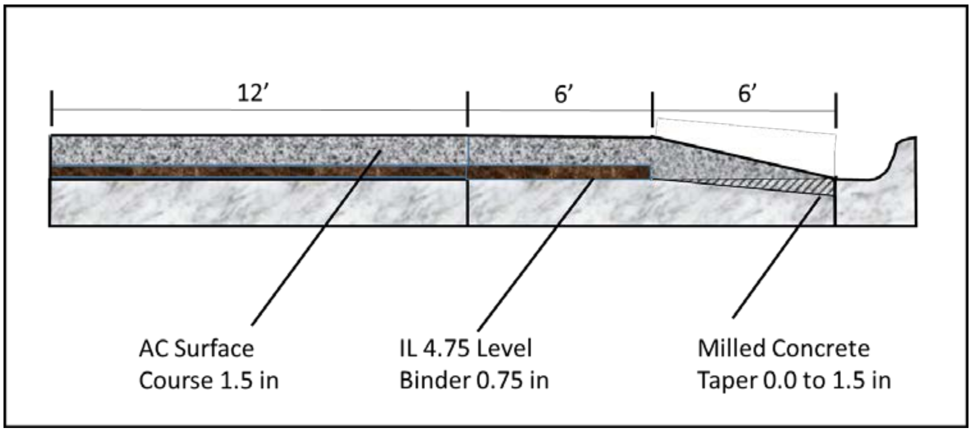


Figure 4.3. Typical milling taper, level binder, and AC surface detail at gutter in urban section for existing concrete pavement.

When the existing urban curb-and-gutter cross-section was a composite pavement (AC over PCC pavement), the existing AC was milled to the thickness of the overlay, a level binder placed full-width gutter to gutter, and then the surface placed as shown in Figure 4.4.

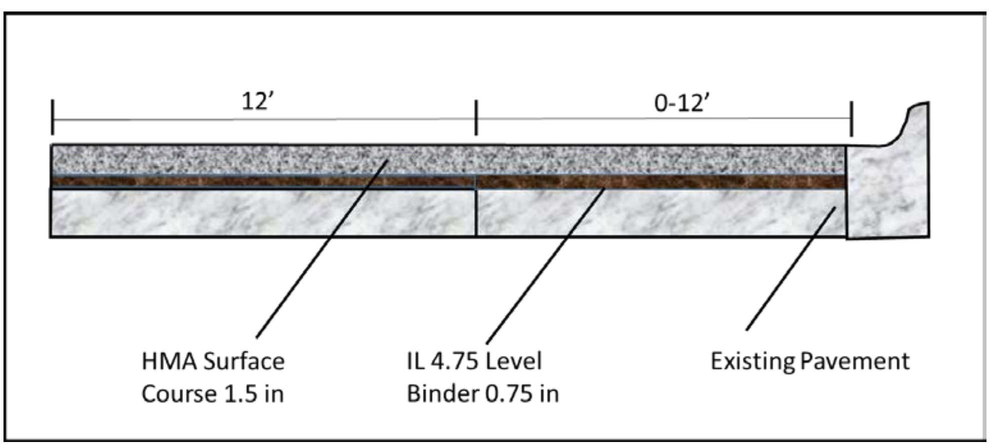


Figure 4.4. Typical milling, level binder, and AC surface detail at gutter in urban section for composite pavement.

For rural sections, with ditch drainage, the outside 12 in of the milled surface were not covered with level binder. The result was that the surface thickness at the outside edge (12 in) was 2.25- to 2.5-in thick, depending upon the project, as shown in Figure 4.5. Figure 4.6 presents a typical photo of the rural outside-12-in detail, as seen on US 52 (IL 53 to Laraway Road—60Y02). A summary of the cross sections is provided in Appendix A. The actual plan cross-section details are available in the interim reports for each project in the study (Lippert et al. 2016, 2017).

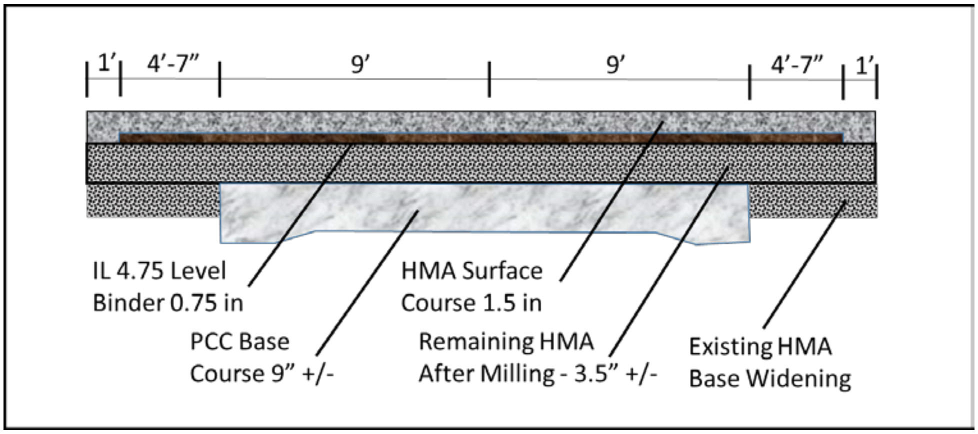


Figure 4.5. Typical existing AC, surface milling, level binder, and AC surface detail in rural section.

As part of construction monitoring activities in this study, paving maps were developed for each project to memorialize the mat-construction sequence. The main purpose for doing this was to have a record of confined and unconfined edge conditions. Edge-confinement conditions impact the level of density that can be obtained at the edge of the paving mat. Unconfined edges tend to result in lower density and have the potential to contain highly permeable pavement at the edge of the mat. Low-density areas have a higher potential to develop raveling and potholes over time. Detailed paving-sequence maps and roller information for level binder placement were presented in interim reports previously (Lippert et al. 2016, 2017).

For compaction effort of the level binder, two static, three-wheeled rollers were used as breakdown and intermediate rollers. A vibratory roller in static mode was used as a finish roller. Figure 4.6 shows the roller type used for breakdown and intermediate rolling, while Figure 4.7 shows the finish roller used on the level binder.

4.5 TACK COAT ON LEVEL BINDER

Prior to placement of the surface course, the level binder was cleaned and then tacked. Figure 4.8 shows Crawford Avenue/Pulaski Road (60Y03), where traffic was not allowed on center-turn lanes, and resulting “zebra” stripes. This case of striping was the most extreme observed, traffic at other projects spread the tack coat to some degree by the time of paving. Figure 4.9 shows US 52 (Gougar Road to Second Street—60N07), which was under traffic and had a fairly uniform tack coat application.

Figure 4.6. US 52 (60Y02) level binder construction with exposed, milled 12 in at edge.

Figure 4.7. Curb-and-gutter section of Washington St. (60Y04) level binder construction and milling at gutter edge.

Figure 4.8. Crawford Avenue/Pulaski Road (60Y03) tack coat of level binder in center lane without traffic.

Figure 4.9. US 52 (Gougar Road to Second Street—60Y07) tack coat of level binder under traffic.

4.6 SURFACE COURSE

The main experimental feature of this study was to monitor and compare performance differences of AC surfaces that IDOT had placed in construction projects on the June 13, 2014, letting. In all, five projects were built with eight different AC surface courses. Variables included RAS ranging from 0 to 5% of the AC mix and RAP ranging from 5 to 39% of the AC mix. Levels of ABR ranged from 15 to 48%. The virgin asphalt binder PG grade was varied, with grades of PG 52-34, PG 58-34, PG 58-28, and PG 64-22 being used. All AC mixes were N-70 mixes.

Unique to the study was the use of a total-recycle asphalt (TRA) surface course mix on two of the projects on US 52 (60N07 and 60N08). The key features of TRA are the use of all recycled aggregates (RAP, slag, or crushed concrete) in conjunction with ABR allowances up to 60% from both fractionated RAP (screened and sized RAP, termed FRAP) and RAS. The contractor was allowed only recycled aggregate options to choose from for meeting AC mix volumetric requirements. TRA designs were N70 gyratory mixes, with an air void target value of 3%, rather than the traditional 4%.

Other than the different AC mixes being used, the construction would be considered typical of construction in Region 1/District 1. Paving of the surface course resulted in different thicknesses, depending upon rural or urban cross sections due to how the level binder and curb details were handled, as shown in Figures 4.3 and 4.4 above. Paving of the 12-in-wide surface “step” is presented in Figure 4.10 below. Details of the AC mixes and specifications were previously provided in the interim reports (Lippert et al. 2016, 2017).

Details of the AC surface-mix paving experiment are provided in Table 2.1. Note that some projects have head-to-head comparisons of AC mixes by direction or end-to-end by key variable changes, such as RAS/no RAS and PG binder grade, while keeping constant other factors, such as ABR and PG grade. Although not part of the overall plan, comparisons of thick- and thin-pavement families also resulted.

For all projects, the surface course thickness was 1.5 in, using typical paving equipment and procedures; however, it was noted that the paver used a 24-ft reference ski rather than a 30-ft ski as a leveling reference. The ski was located on the left side of the paver for the construction of all lanes under study. The right side of the paver did not have a reference and was adjusted from time to time to control material yield. After mat laydown, compaction was achieved with two dual-drum vibratory rollers, followed by a dual-drum vibratory roller operated in static mode as the finish roller. All the projects followed this general paving-train pattern.

Figure 4.10. Surface course paving of rural cross-section on US 52 (US 53 to Laraway 60Y02).

The surface was a nominal 1.5 in, the level binder detail (shown in Figures 4.3 and 4.5 above) resulted in the surface layer having a transverse "step" detail in the level binder. This detail seemed to be the cause of limited hairline cracking at the location of the "step" and slight segregation where additional material is supplied to the area at the outside edge of the screed, as seen in Figure 4.11. Additional surface course construction details were previously presented in the interim reports for each project (Lippert et al. 2016, 2017).

Figure 4.11. Evidence of hairline crack at underlying transition from level binder to no level binder. Level binder (4.75 mm) to left of crack and milled surface to right of crack. Increased segregation also at pavement edge (right side of photo).

CHAPTER 5: POST-CONSTRUCTION CONDITIONS

After construction of the various sections, an effort was made to capture the post-construction surface conditions of the resulting pavement-improvement soon after completion and monitor performance for several years on an annual basis. This task was accomplished by high-speed profilograph testing and crack surveys.

5.1 POST-CONSTRUCTION CRACK SURVEYS

For both the 2013 and 2014 let projects, IDOT conducted post-construction condition surveys. These detailed surveys were presented in previous project reports (Lippert et al. 2016, 2017). Little to no distress was noted post-construction, but the surveys did confirm the pavement condition at this stage. Of note was that cracking distress as pointed out in Figure 4.11 was not observed and recorded in the surveys. This result may be because cracks that did appear were very difficult to identify visually from the shoulder during the survey, especially those in the far lane. Also, the location of the level binder edge is within the main pavement structure, a location where reflective cracking is not expected. In conducting a final field review, it was found that this detail could not be tied to pavement distress. The main distresses found post-construction were the start of transverse cracks and limited centerline joint distress. The limited distress data captured are included in the performance and analysis chapters that follow.

5.2 PROFILE TESTING

Post-construction profile testing, as well as monitoring throughout this project, was performed by ERI, Inc., using a five-point laser and video-log system. This system provides IRI (1/4-car simulation) and rut depth of each wheel path, as well as video. Although the video was not used to monitor the pavement condition, it was used as a reference to ensure consistency in data collection. Station referencing was critical for comparing results throughout the study. To ensure accurate stationing, temporary physical references (reflective cones) were positioned at the start and finish of each data run to provide a signal to trigger data-recording start at a known station.

After the data were collected, the dataset was reviewed and summaries developed, averaging the collection over 0.1- and 0.01-mi increments. For project monitoring, only the 0.01 mi increments were used for data summaries and analysis. Summaries were developed by project segment, by direction, and overall for the project. However, prior to usage, the data were screened.

The timing of detailed profile data collection on the 2013 let projects was such that two winters had elapsed before data was collected. Post-construction IRI can be assumed to be less for the 2013 projects than the data collected in 2015. Rutting can only be assumed to be near zero for as-built conditions for the 2013 let projects.

5.2.1 Initial pavement smoothness

For all the projects in this study, Article 406.11 "Surface Test of the Standard Specification" was the controlling specification for pavement smoothness (IDOT 2012b). The main testing device was a simple wheeled beam with an adjustable bolt as the indicator. The straightedge is rolled along with the depth setting at the 3/16-in specification limit. Any surface scratch marks indicate an area out of specification. Variations not exceeding 3/16 in for mainline pavement are allowable and accepted. Variations over 3/16 in result in a surface course mixture tonnage deduction. However, if the variation equals or exceeds ¾ in, the area affected must be removed and replaced.

In the mid-1990s, IDOT developed an incentive/disincentive surface test for new full-depth AC, new PCC pavement, and new Interstate overlays to achieve smoother pavements. As a result of the incentive/disincentive, ride quality on Interstate pavements and new pavement construction greatly improved. Ride quality is additionally assisted on Interstates by the required use of material-transfer devices when paving AC. In chapter 53 of the IDOT *Design and Environment Design Manual* (IDOT 2017), a discussion on IRI and rutting data is presented. The *Design Manual* notes that pavement IRI values over 175 in/mi are considered unacceptable. Table 5.1 presents a quartile analysis of IRI data on the entire Illinois Interstate system. Although the incentive/disincentive specifications do not apply to the work in this study, the information does provide a frame of reference for what smoothness values could potentially be achieved.

Table 5.1. Quartile Analysis of IRI Data on Illinois Interstates

Quartile	IRI Value (in/mi)
First (Smoothest)	IRI = 60
Second	60 ≤ IRI ≤ 75
Third	76 ≤ IRI ≤ 100
Fourth	IRI > 100

Note: IRI values over 175 in./mi are considered unacceptable.

Table 5.2 presents the overall profile test results for the investigated projects, filtered as noted above. The IRI datasets show that the overlaid projects after construction were rougher than in-service Interstate pavements. This finding would indicate a potential for improvement in smoothness of newly constructed overlays. The benefit of smoother surfaces must be balanced with the cost of achieving this goal on low-speed/low-volume facilities, such as several roadways included in this study.

Although smoother pavements are desired, achieving this end is more challenging for non-Interstate highways. Interstate highways tend to receive rehabilitation when the pavement is in better and smoother condition, benefit from use of material-transfer devices, and are covered under a smoothness-incentive specification. Less critical roadways are often allowed to develop more distresses and roughness prior to rehabilitation.

Figure 5.1 presents the relationship for the 2014 let projects of pre-construction IRI versus post-construction IRI. It is seen that the condition of the pavement has a limited impact on the resulting initial ride of the new pavement surface.

In reviewing the IRI datasets by wheel path, it was noted that the right-wheel-path was consistently rougher than the left wheel path on all the projects under study. The difference in wheel path data was also noted in other data sets in Illinois and other states.

Table 5.2. Initial Measured Pavement-Profile Data Summary

Project	Contract	Direction	Test Date	IRI, in/mi			Rut Depth, in		
				Left WP	Right WP	Ave	Left WP	Right WP	Ave
26th St.[1]	60L62	Both	11/4/15	141	149	145	0.04	0.04	0.04
Harrison St.[1]	60B67	Both	11/4/15	128	155	141	0.03	0.04	0.04
Richards St.[1]	60P70	Both	11/3/15	108	163	136	0.06	0.05	0.05
Wolf Rd[1]	60M30	Both	11/4/15	86	110	98	0.02	0.03	0.02
Crawford Ave/Pulaski Rd[2]	60Y03	NB	12/17/14	103	166	134	0.01	0.01	0.01
Crawford Ave/Pulaski Rd[2]	60Y03	SB	12/17/14	102	164	133	0.01	0.01	0.01
US 52—IL 53 to Laraway[2]	60Y02	EB	12/17/14	78	90	84	0.01	0.01	0.01
US 52—IL 53 to Laraway[2]	60Y02	WB	12/17/14	73	86	80	0.01	0.01	0.01
US 52—Laraway to Gougar[3]	60N08	Both	11/3/15	83	93	88	0.02	0.02	0.02
US 52—Gougar to Second St.[3]	60N07	Both	11/3/15	64	83	74	0.01	0.02	0.01
Washington St.[3]	60Y04	EB	11/3/15	75	89	82	0.01	0.01	0.01
Washington St.[3]	60Y04	WB	11/3/15	86	94	90	0.01	0.02	0.02

[1] 2013 let projects did not receive detailed profile testing until 2015, two winters after construction.

[2] 2014 let project tested after construction in 2014, average of all segments.

[3] 2014 let project tested after construction in 2015, average of all segments.

For paving, the contractor placed the reference ski on the left side of the paver, which is typical of AC construction in Illinois and nationally. For these projects, this reference ski was the only one used on the paver. After reviewing the data, a follow-up discussion with the paving contactor for the 2014 let projects found that to adjust yield, the cross-slope would be slightly adjusted from time to time and may have contributed partly to the ride quality differences. The research team also suspected that the level binder and gutter-milling detail, as shown in Figures 4.4 and 4.5, above, contributed to some right-wheel-path roughness. The difference was also seen in the passing lane, where this edge detail would not have been a factor.

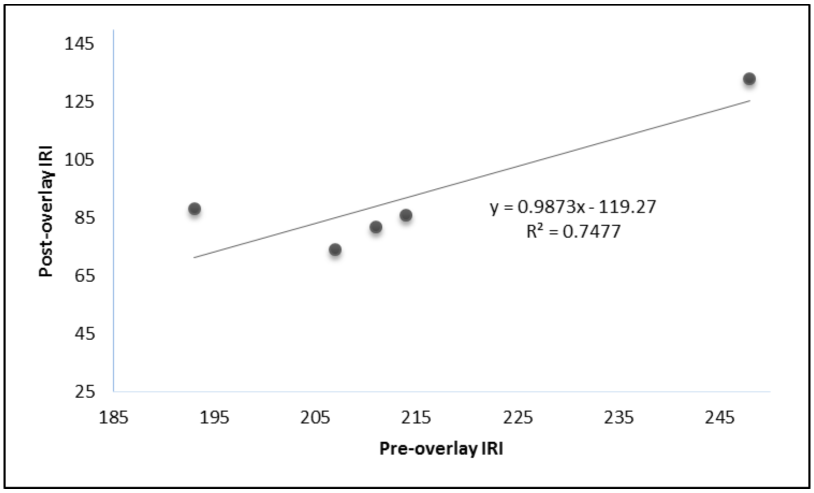


Figure 5.1. Pre-construction IRI's impact on post-construction IRI for 2014 let projects.

After reviewing the data and possible causes for wheel path ride differences, it was concluded that the data are representative of the pavements tested. Efforts to improve the smoothness in the right-wheel-path would improve the overall ride quality of the pavements. Cross-section modification (extending level binder to the edge of pavement) and an additional ski control would have positive benefits for right-wheel-path smoothness. Revised specification controls are another method to influence more directly the resulting pavement smoothness at construction; however, the cost and benefits of such requirements need to be determined.

5.2.2 Rutting after Construction

Rutting summaries are presented in Table 5.2 for the 2013 and 2014 let projects. The 2013 let project data collection was approximately two years after construction of these and does not represent a true initial rut depth. The 2014 let projects were tested a few weeks to months after construction to capture the initial rut depths. The data show that wheel path rutting was in the range of 0.01 to 0.02 in. It is suspected that much of the measured rut depth on the 2014 let projects was created immediately after paving, when traffic was allowed on the mat prior to its being fully cooled. Detailed rutting has been previously reported, with the most current result found in Appendix C.

5.2.3 Profile Trends with Time

Due to differing construction times, testing dates and age of the various pavement segments, the data were normalized to present the change in IRI and change in rut depth on an annual basis. For the 2013 let projects the first detailed data on IRI and rutting was collected approximately two years after construction. For IRI change per year determination, the 2015 IRI data values and final IRI data collected in 2017 were used to determine the average annual change in IRI. The resulting data is the change in IRI for the last two years of project life. For the 2014 let projects, initial profile data was collected a few weeks to months after completion of the projects in 2014 and 2015, thus representing the life of the pavement. Table 5.3 presents the final summary IRI values with annual rate of IRI change per year.

The change in IRI on an annual basis is remarkably similar for all projects, with the average increase in IRI being 4 in/mi annually. A few minor negative trends in IRI could be attributed to data collection location variability.

Table 5.3. Final Pavement IRI Data Summary with Rate of Change

Project	Contract	Direction	Test Date	IRI, Inches/Mile			Average Annual Change in IRI, Inches/Mile/Year		
				Left WP	Right WP	Ave	Left WP	Right WP	Ave
26th St.[1]	60L62	Both	4/26/17	145	148	146	2.7	-0.7	0.7
Harrison St.[1]	60B67	Both	4/26/17	135	167	151	4.7	8.0	6.7
Richards St.[1]	60P70	Both	4/25/17	113	174	144	3.3	7.3	5.3
Wolf Rd[1]	60M30	Both	4/26/17	93	116	105	4.7	4.0	4.7
Crawford Ave/Pulaski Rd[2]	60Y03	NB	4/25/17	112	172	142	3.9	2.6	3.4
Crawford Ave/Pulaski Rd[2]	60Y03	SB	4/25/17	114	172	142	5.1	3.4	3.9
US 52—IL 53 to Laraway[2]	60Y02	EB	4/25/17	88	102	95	4.3	5.1	4.7
US 52—IL 53 to Laraway[2]	60Y02	WB	4/25/17	85	99	92	5.1	5.6	5.1
US 52—Laraway to Gougar[3]	60N08	Both	4/25/17	79	90	85	-2.7	-2.0	-2.0
US 52—Gougar to Second St.[3]	60N07	Both	4/25/17	71	100	85	4.7	11.3	7.3
Washington St.[3]	60Y04	EB	4/25/17	76	91	83	0.7	1.3	0.7
Washington St.[3]	60Y04	WB	4/25/17	86	97	91	0.0	2.0	0.7

[1] 2013 let projects did not receive detailed profile testing until 2015, two winters after construction.

[2] 2014 let project tested after construction in 2014, average of all segments.

[3] 2014 let project tested after construction in 2015, average of all segments.

Annual change in rut depth was calculated assuming the pavement was built with no rutting and that the final measured rut depth occurred over the life of the pavement. Table 5.4 present the final summary rutting values with annual rate of rutting increase per year. Richards Street shows a higher annual rutting change than other sections. This trend is tied to the amount and severity of alligator cracking found on this project. In general, all sections have no rutting issues to date.

Smoother pavements at construction would result in better pavement ride throughout the life of the pavement. Assuming these pavements would serve a minimum 10 year life, an additional

40 in/mi would be expected to be added to the as-built IRI on these sections. Such a change in IRI would result in many of these projects being considered rough.

Making a similar life assumption considering pavement rutting, Richards Street will likely need parts of pavement to be repaired. As noted above, there are currently alligator cracked areas indicating a need for repair in the near future. Assuming the rutting rate does not increase in other pavement sections, repairs due to rutting may not be needed.

Table 5.4. Final Pavement Rutting Data Summary with Rate of Change

Project	Contract	Direction	Test Date	Rut Depth, in			Average Annual Change in Rut Depth in/yr[1]		
				Left WP	Right WP	Ave	Left WP	Right WP	Ave
26th St.	60L62	Both	4/26/17	0.05	0.04	0.04	0.014	0.011	0.011
Harrison St.	60B67	Both	4/26/17	0.04	0.06	0.05	0.011	0.017	0.014
Richards St.	60P70	Both	4/25/17	0.07	0.10	0.09	0.020	0.029	0.026
Wolf Rd	60M30	Both	4/26/17	0.03	0.02	0.02	0.009	0.006	0.006
Crawford Ave/Pulaski Rd	60Y03	NB	4/25/17	0.03	0.03	0.03	0.013	0.013	0.013
Crawford Ave/Pulaski Rd	60Y03	SB	4/25/17	0.02	0.02	0.02	0.009	0.009	0.009
US 52—IL 53 to Laraway	60Y02	EB	4/25/17	0.03	0.01	0.02	0.012	0.004	0.008
US 52—IL 53 to Laraway	60Y02	WB	4/25/17	0.04	0.01	0.02	0.016	0.004	0.008
US 52—Laraway to Gougar	60N08	Both	4/25/17	0.03	0.02	0.02	0.018	0.012	0.012
US 52—Gougar to Second St.	60N07	Both	4/25/17	0.02	0.01	0.02	0.011	0.006	0.011
Washington St.	60Y04	EB	4/25/17	0.01	0.01	0.01	0.005	0.005	0.005
Washington St.	60Y04	WB	4/25/17	0.01	0.01	0.01	0.005	0.005	0.005

[1] An Initial rut depth of 0.0 was used to determine the annual change in rut depth for all projects.

CHAPTER 6: MATERIALS TESTING

6.1 INTRODUCTION

This chapter summarizes the results of testing performed by Illinois Center for Transportation (ICT) and IDOT's Bureau of Materials and Physical Research (BMPR) teams. Table 2.1 presents testing performed in this study and the laboratory responsible for conducting the test. The testing results include (1) basic mix design verification: virgin asphalt binder, asphalt binder content, and aggregate gradation; (2) mechanical properties: Marshall stability, Cantabro loss, tensile strength ratio (TSR), Texas overlay, complex modulus, flow number, beam fatigue, creep compliance/indirect tensile test (IDT) strength, Hamburg wheel-tracking (HWT), and Illinois flexibility index test (I-FIT). Field cores were also collected for three years for conducting creep compliance/IDT strength, Hamburg WT, and I-FIT tests on surface mixes. Detailed testing data can be found in Appendix D.

6.2 MIX DESIGN VERIFICATION

Based on the AC mix-design verification test results, the key observations are as follow:

- All neat asphalt binders satisfy the requirement of AASHTO M 332. The detailed binder-test results can be found in Appendix D.1.
- The extracted aggregate gradation for all AC mixes sampled from the plant is consistent with the job-mix formula (JMF).
- Several of the asphalt binder extractions determined for plant AC mixes differed from the JMF significantly, as follows: 177M (0.8% higher than JMF), 140M (0.3% lower than JMF), 185N08 (0.4% lower than JMF), 163N08 (0.3% lower than JMF), and 147M (0.3% higher than JMF). It should be noted that asphalt binder content affected the performance of AC mixtures, which is discussed later for each test. The detailed test results can be found in Appendix D.1.

6.3 TESTING RESULTS

To better illustrate the effect of AC mix-design parameters on mechanical properties, AC mixes were divided into surface mix and level binder mix. The surface mixes with similar mix designs were grouped as shown in Figure 6.1.

6.3.1 Marshall Stability Results

Two samples were fabricated (Marshall-compacted and gyratory-compacted) for each mix type to evaluate the effect of compaction (specimen configuration) on Marshall stability. Figures 6.2(a) and (b) present the Marshall stability test results for Marshall-compacted specimens and gyratory-compacted specimens, respectively.

In Group G1, the 159Y04 mix with PG 58-34 had stability comparable to the 159Y02 with PG 58-28 binder, indicating that one grade difference in low performance grade (PG) may not significantly affect AC mixture's stability. In Group G2, higher ABR resulting from higher RAS and RAP content (157Y03 mix) caused lower stability. It was also noted that the 157Y03 mix had lower PG for both high and low temperatures. In Group G3, asphalt binder played a significant role in stability. Higher AC results in lower stability. For Group G4 in which both AC mixes have high ABR, they seem to have lower stability than the AC mixes in other groups. Mixes in Group G5 used polymer-modified binders; however, because of high AC content, the stability of the level binder course mixes was lower than that of other mixes.

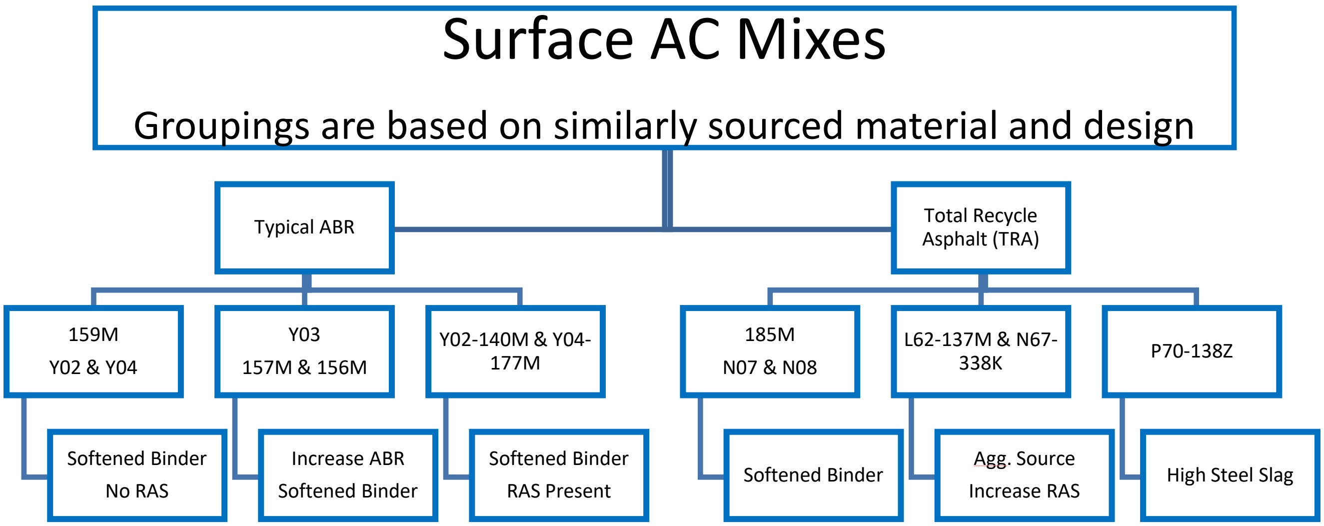
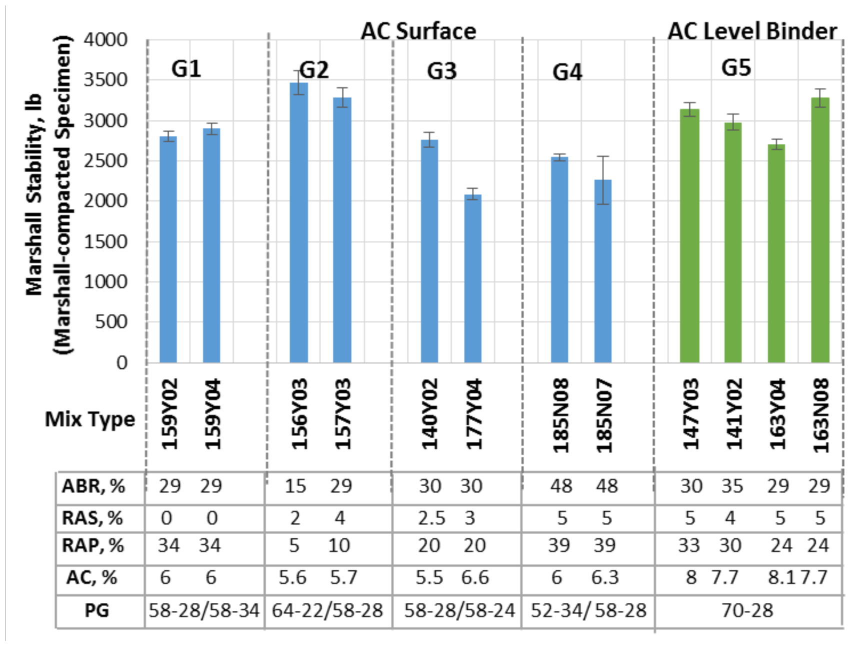


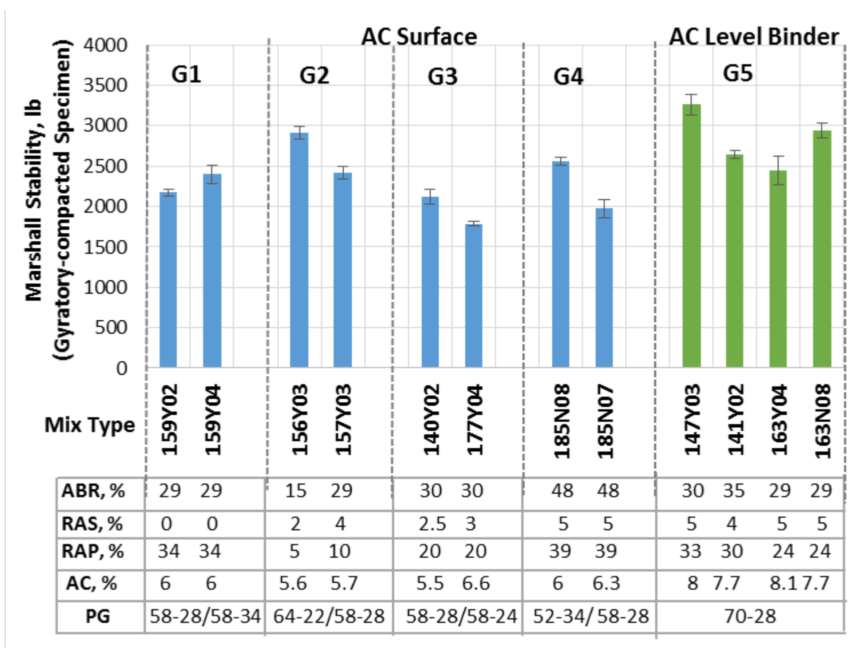
Figure 6.1. Grouping of surface AC mixes.

The most significant design parameters for Marshall stability in this study were asphalt binder content and ABR. Higher asphalt binder content, and/or higher ABR may result in lower Marshall stability. The same trend was noted for both Marshall- and gyratory-compacted specimens.

Because the availability of Marshall equipment is limited due to the adoption of SuperPave® mix design procedures in the 1990s, an effort was undertaken to compare the Marshall stability of traditional hammer-compacted, 4-in Marshall specimens to more available gyratory-compacted specimens that were cored, resulting in 4-in specimens suitable for conducting a stability test. For this effort, the N50 (2013 let projects) and N70 (2014 let projects) mixes were compacted to 4 +/- 0.5% air voids using both gyratory and Marshall hammer compactors. This process required the number of blows of the Marshall hammer to be varied from 25 to 105, depending upon the mix, to obtain the proper air void. Figure 6.3 plots the Marshall stability of a Marshall-compacted specimen with that of a gyratory-compacted specimen that was cored (to produce the standard 4-in Marshall-size specimen). As shown, the Marshall stability of the gyratory-compacted specimen was biased lower than that of the Marshall-compacted one. This difference is explained by the facts that the compaction effort (energy) by the Marshall apparatus is higher than that of gyratory compactor and that the gyratory compactor was developed to better simulate field compaction.



(a)



(b)

Figure 6.2. Marshall stability: (a) Marshall-compacted specimens and (b) gyratory-compacted specimens.

In general, all AC mixes met the Marshall stability requirements. The level binder mix was expected to have a lower stability value but because of using RAS and PG70-28, the values were the highest.

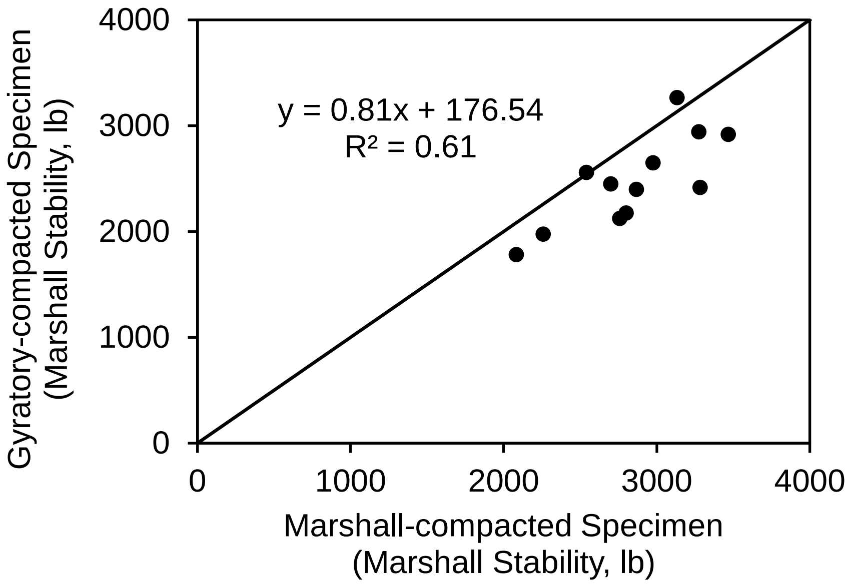


Figure 6.3. Marshall stability between Marshall-compacted and cored gyratory-compacted specimens at 4+/- 0.5% air voids.

6.3.2 Cantabro Loss Test Results

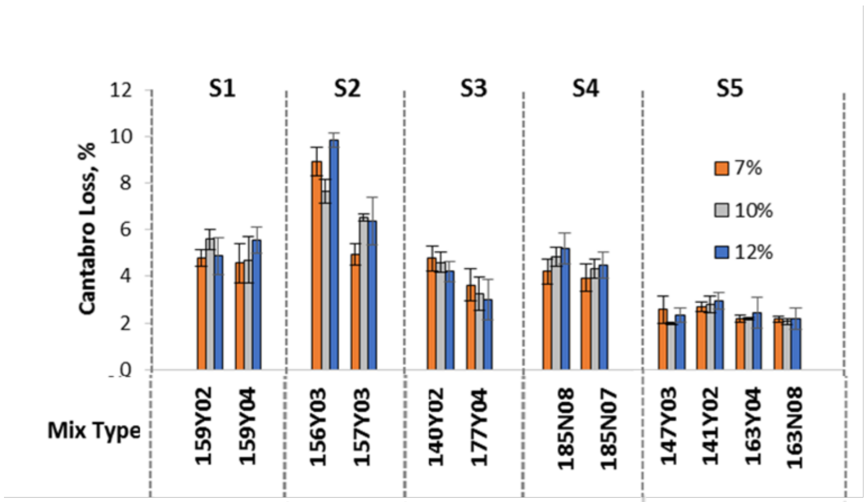
The Cantabro loss test was used to characterize durability of the AC mixes. Figure 6.4 shows the Cantabro loss for each AC mix type for three air void contents. Overall, the Cantabro loss was less than 10% regardless of AC mix type. Previous studies on open-graded friction course (OGFC) mix showed that the Cantabro loss ranged from 12% to 31% (Punith et al. 2012). A study by Doyle and Howard (2010) on a 9.5-mm, dense-graded Mississippi AC mixture showed that the Cantabro loss ranged from 2.8 to 11.7%. The AC mixes in the current study are also 9.5-mm, dense-graded; thus, a low Cantabro loss value was expected for the dense-graded AC mixes.

Group S5 had the lowest Cantabro loss, which is due to polymer-modified binders and 4.75-mm aggregate gradation. Mixes within Group S5 were comparable in Cantabro loss. The 156Y03 mix had the highest Cantabro loss, which may imply that asphalt binder grade and binder content plays an important role in keeping the cohesiveness of AC.

The level binder has the lowest lost due to having the highest binder content. For AC surface mixes, the higher Marshall stability correlated to greater Cantabro loss suggesting a potential brittle AC mix.

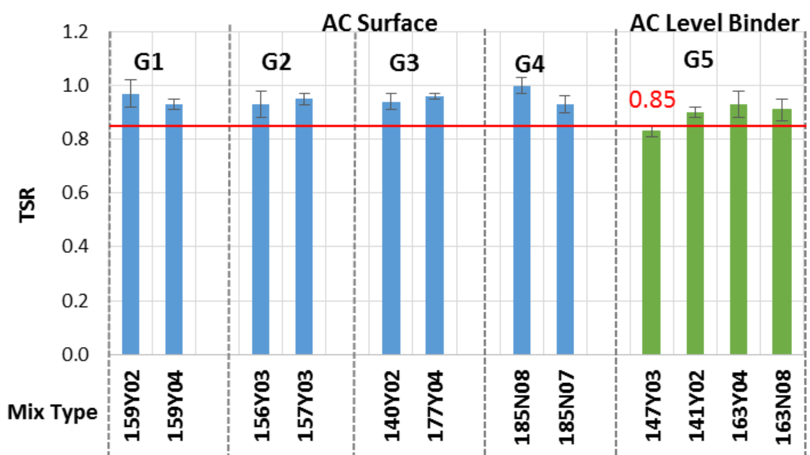
6.3.3 Moisture-Damage Test Results (TSR)

The moisture-damage resistance of AC mixtures was characterized by the IL-Modified AASHTO T 283 TSR test. Figure 6.5 presents the TSR for each mix. As shown, all AC mixes had acceptable ratios, except that the TSR value for the 147Y03 mix was slightly below the threshold value of 0.85. It was verified in the JMF that the 147Y03 mix passed the TSR requirement. This test, although may provide an indication of moisture susceptibility, it may not distinguish cracking potential of AC mixes.



	159Y02	159Y04	156Y03	157Y03	140Y02	177Y04	185N08	185N07	147Y03	141Y02	163Y04	163N08
ABR, %	29	29	15	29	30	30	48	48	30	35	29	29
RAS, %	0	0	2	4	2.5	3	5	5	5	4	5	5
RAP, %	34	34	5	10	20	20	39	39	33	30	24	24
AC, %	6	6	5.6	5.7	5.5	6.6	6	6.3	8	7.7	8.1	7.7
PG	58-28/58-34		64-22/58-28		58-28/58-24		52-34/ 58-28		70-28			

Figure 6.4. Cantabro loss test results.



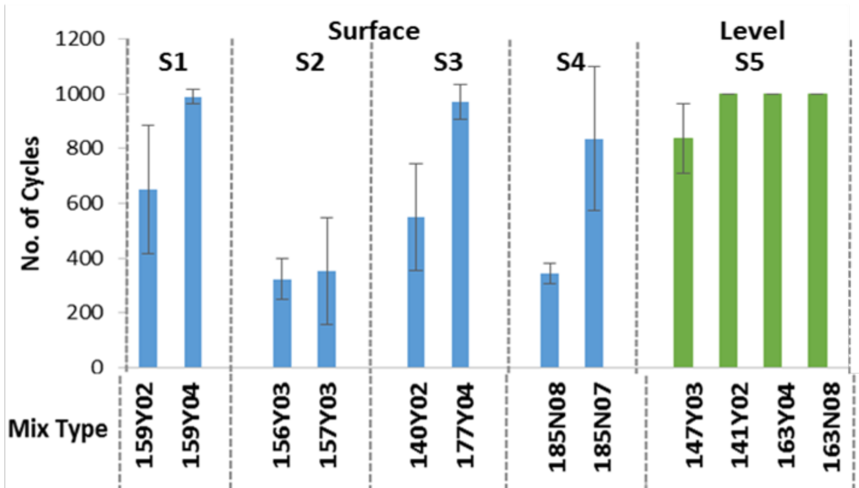
	159Y02	159Y04	156Y03	157Y03	140Y02	177Y04	185N08	185N07	147Y03	141Y02	163Y04	163N08
ABR, %	29	29	15	29	30	30	48	48	30	35	29	29
RAS, %	0	0	2	4	2.5	3	5	5	5	4	5	5
RAP, %	34	34	5	10	20	20	39	39	33	30	24	24
AC, %	6	6	5.6	5.7	5.5	6.6	6	6.3	8	7.7	8.1	7.7
PG	58-28/58-34		64-22/58-28		58-28/58-24		52-34/ 58-28		70-28			

Figure 6.5. Tensile Strength Ratio (TSR) test results.

6.3.4 Texas Overlay Test Results

The Texas overlay tester (OT) was used to evaluate cracking resistance of AC mixtures. The number of cycles to failure was obtained in this test when the initial load was reduced by 93% (Zhou et al. 2005). Figure 6.6 presents the number of cycles to failure from the OT for each AC mix. The variation in OT results for most surface mixes was high, and the coefficient of variation (COV) among five replicates could be as high as 56% for the 157Y03 mix. However, the OT seems to distinguish qualitatively the AC mixes in Groups S1, S3, and S4. The soft, low-PG

(Group 1) and high asphalt binder content (Groups 3 and 4) were the mixes achieving a higher number of cycles to failure (i.e., better cracking resistance). As the asphalt binder content increases so does the number of cycles to failure. The use of a softer asphalt binder also increased the number of cycles to failure. For level binder mixes (Group S5), the number of cycles were all high because of the polymer-modified binder used and the higher AC.



	S1		S2		S3		S4		S5			
Mix Type	159Y02	159Y04	156Y03	157Y03	140Y02	177Y04	185N08	185N07	147Y03	141Y02	163Y04	163N08
ABR, %	29	29	15	29	30	30	48	48	30	35	29	29
RAS, %	0	0	2	4	2.5	3	5	5	5	4	5	5
RAP, %	34	34	5	10	20	20	39	39	33	30	24	24
AC, %	6	6	5.6	5.7	5.5	6.6	6	6.3	8	7.7	8.1	7.7
PG	58-28/58-34		64-22/58-28		58-28/58-24		52-34/ 58-28		70-28			

Figure 6.6. Texas overlay test results.

6.3.5 Dynamic Modulus Test Results

The main purpose of this test is to determine the viscoelastic behavior of AC under repeated loading, by assessing the stress-to-strain relationship of the material subjected to a continuous sinusoidal loading. The main output of the test is the dynamic modulus (|E*|) and phase angle (δ). The test followed AASHTO standard T342, in which cylindrical specimens of 4-in diameter and 6-in height were cored from gyratory-compacted specimens and tested across five temperatures, 14°F, 39°F, 70°F, 100°F, and 129°F; and under six loading frequencies, 0.1, 0.5, 1, 5, 10, and 25 Hz. Four replicates were tested for each AC mix, and the |E*| and δ values of each mix were taken as the average of the four.

A master curve, for modulus and phase angle, was constructed for each AC mix at a reference temperature of 21°C, following the time–temperature superposition principles. A sigmoidal model was fitted for the modulus master curves following Equation [6.1]; and for the phase angle, a Lorentizan peak function was used for model fitting according to Equation [6.2].

$$\log E^* = \delta + \frac{\alpha}{1+e^{\beta+\gamma * \log f_r}} \quad [6.1]$$

$$\delta = \frac{(a * b^2)}{[\log(f_r)-c]^2+b^2} \quad [6.2]$$

where, δ is the minimum E* value, α is the span of E* values, β and γ are shape parameters of the function, f_r is the reduce frequency, a is the maximum peak value, b is the growth rate, and c is the critical point.

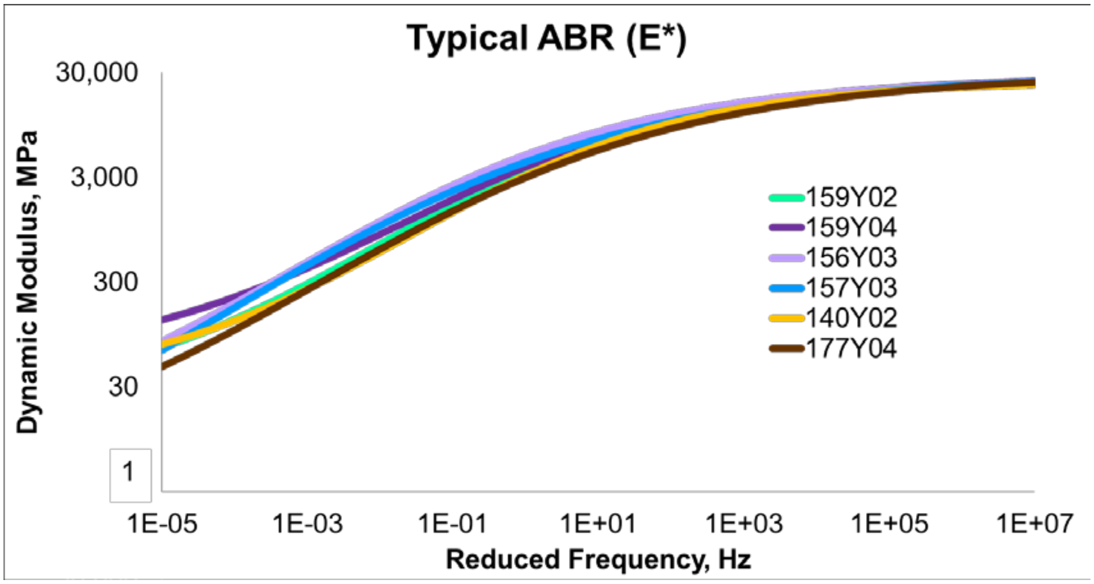
Table 6.1 summarizes the different model coefficients for both modulus and phase angle for all the project mixes.

Table 6.1. Model Coefficients from Dynamic Modulus Test

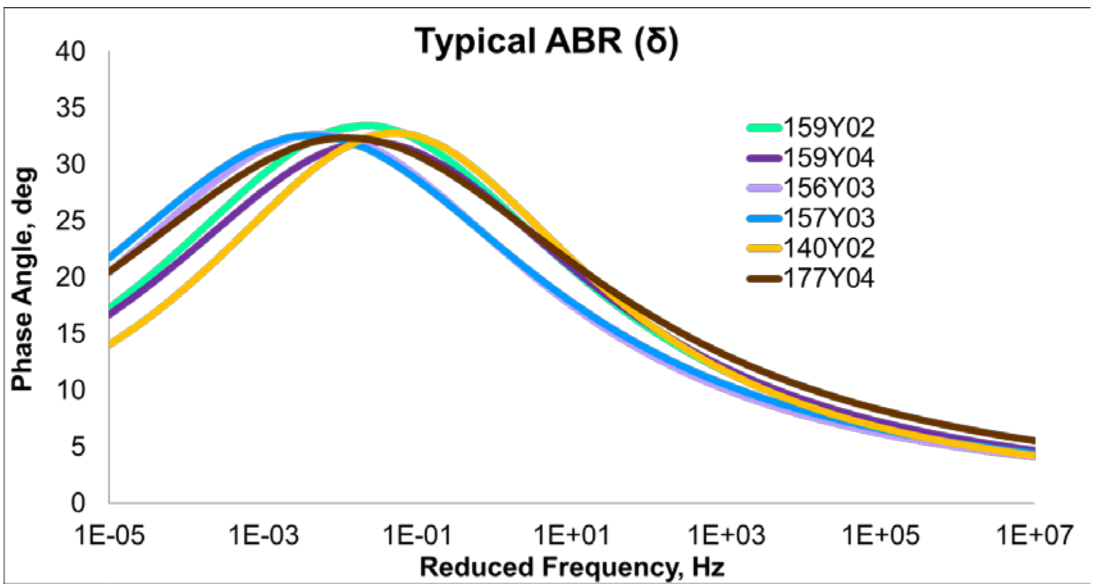
		Sigmoidal Coefficients (E*)				Lorentizan Coefficients (δ)		
		δ	α	β	γ	a	b	c
Typical ABR	159Y02	1.282	3.143	-0.974	-0.495	33.459	3.428	-1.684
	159Y04	1.657	2.781	-0.806	-0.484	31.983	3.558	-1.598
	140Y02	1.449	2.950	-0.818	-0.515	32.800	3.198	-1.302
	177Y04	0.462	4.009	-1.107	-0.390	32.387	4.053	-1.917
	156Y03	0.763	3.674	-1.367	-0.430	32.689	3.526	-2.277
	157Y03	-0.092	4.568	-1.473	-0.359	32.575	3.711	-2.378
TRA	185N07	0.604	3.833	-1.144	-0.392	32.681	3.728	-1.996
	185N08	1.734	2.638	-0.625	-0.466	30.038	3.670	-1.496
	338N67	0.016	4.391	-1.827	-0.289	29.558	4.257	-4.464
	137L62	0.016	4.425	-1.509	-0.345	32.886	3.797	-2.862
	138P70	0.016	4.437	-1.616	-0.409	36.900	3.393	-2.684
Leveling Binders	163Y04	0.910	3.409	-0.821	-0.418	33.917	3.747	-1.667
	163N07	1.078	3.283	-0.798	-0.427	34.270	3.681	-1.761
	163N08	0.651	3.742	-1.030	-0.364	32.803	4.001	-2.124
	141Y02	1.023	3.332	-0.843	-0.406	32.950	3.822	-1.847
	147Y03	0.912	3.406	-0.951	-0.422	34.316	3.721	-2.054

Figures 6.7 (a) and (b) present E* and phase angle master curves for the AC mixes classified as typical ABR. At high frequencies, E* for all AC mixes tends to converge to a narrow range of values around 3,620 ksi, indicating that under low temperature conditions, all the AC mixes have similar elastic behavior. At the lower end of the frequency range, which equates to high temperatures, AC exhibits more viscous behavior, which makes it prone to permanent deformation; under this condition, higher modulus and lower phase angle values are preferred. Mix 159Y04 and, to some extent, mixes 156Y03 and 157Y03 are the ones that show high modulus.

Figures 6.8(a) and (b) present the modulus and phase angle master curves for TRA mixes, respectively. At high frequencies, low temperature behavior, where lower modulus is preferred, all the AC mixes converge close to a value of 3,190 ksi; only mix 138P70 exhibits significantly higher modulus, close to 3,620 ksi; 138P70 is blended with a high amount of steel slag (> 65%) and blended low binder content (5.8%), compared to rest of the TRA mixes. At low frequencies and high temperature, mix 338N67 has consistently higher modulus and lower phase angle, signaling a stronger resistance to permanent deformation. 338N67 contains recycled crushed concrete, which might aid its having a stronger aggregate skeleton.



(a)

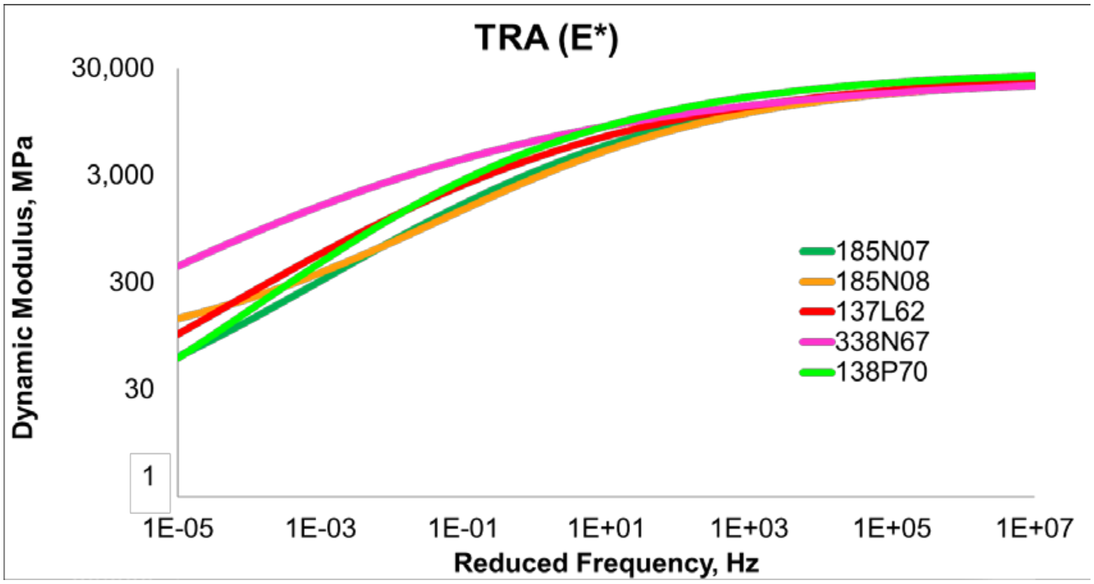


(b)

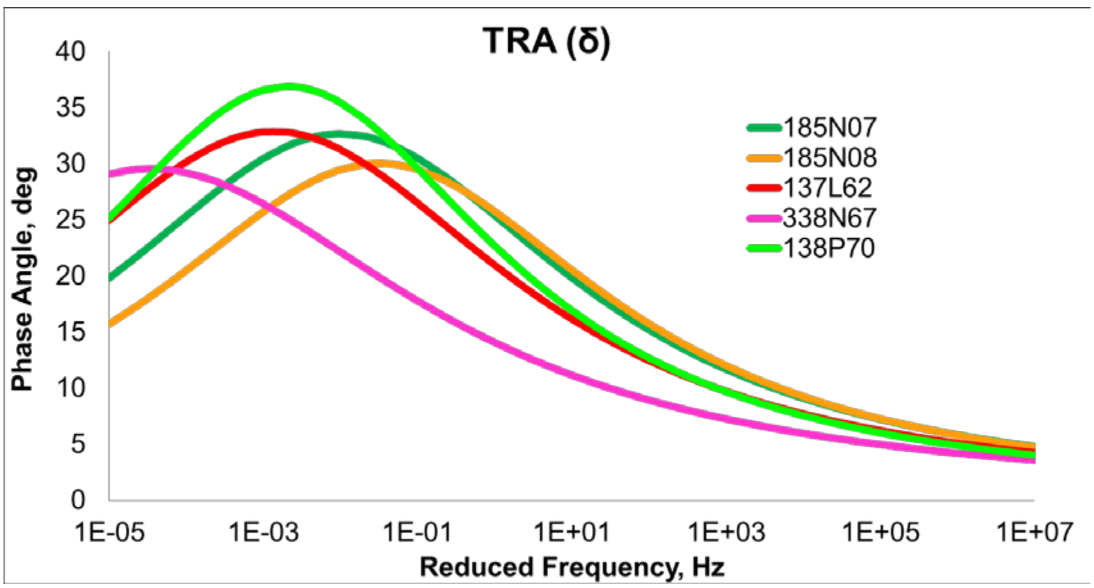
* 1 MPa= 0.145 ksi

Figure 6.7. (a) Dynamic modulus master curves for typical ABR mixes; (b) phase angle master curves for typical ABR mixes.

Figures 6.9 (a) and (b) plot E* and phase angle master curves for level binder AC mixes, respectively. In general, all mixes behave relatively equally across the entire frequency range in both modulus and phase angle. At high frequencies, the modulus of all mixes converges close to 2,750 ksi; and at low frequencies, in a range between 45 and 65 MPa. Although level binders were blended with higher PG asphalt binder (PG 70-28), the high asphalt content of the AC mixes makes them have relatively low modulus, which might aid the purpose of having a flexible mix between old milled pavement and a new surface course.



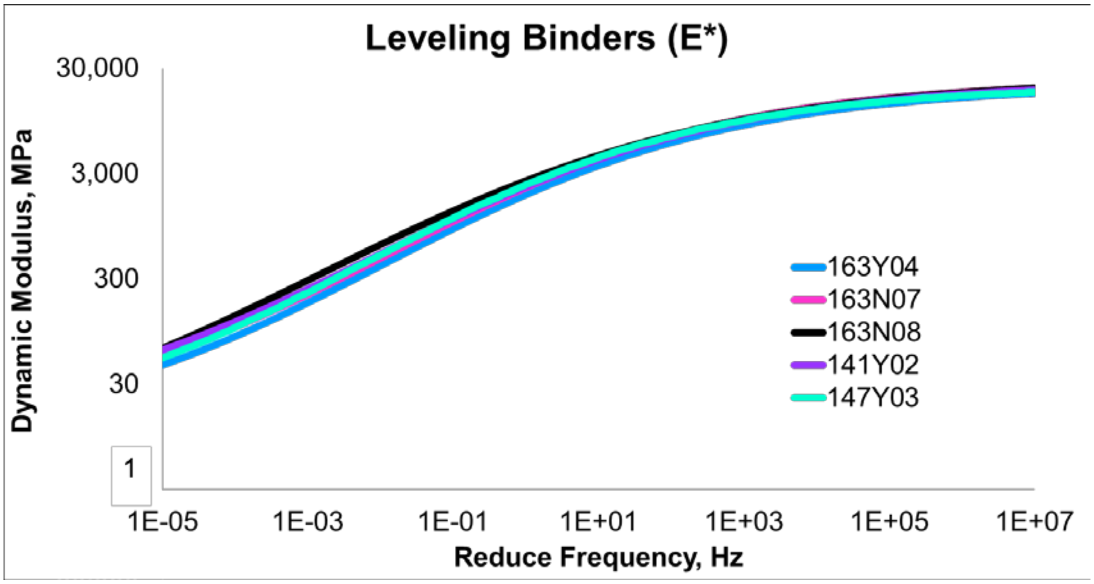
(a)



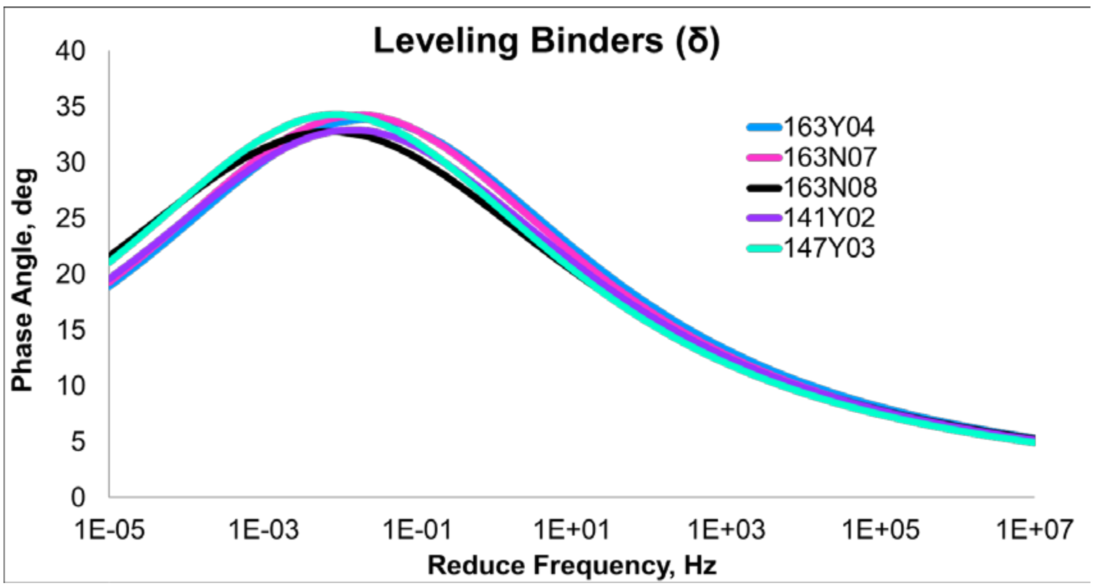
(b)

* 1 MPa= 0.145 ksi

Figure 6.8. (a) Dynamic modulus master curves for TRA mixes; (b) phase angle master curves for typical TRA mixes.



(a)



(b)

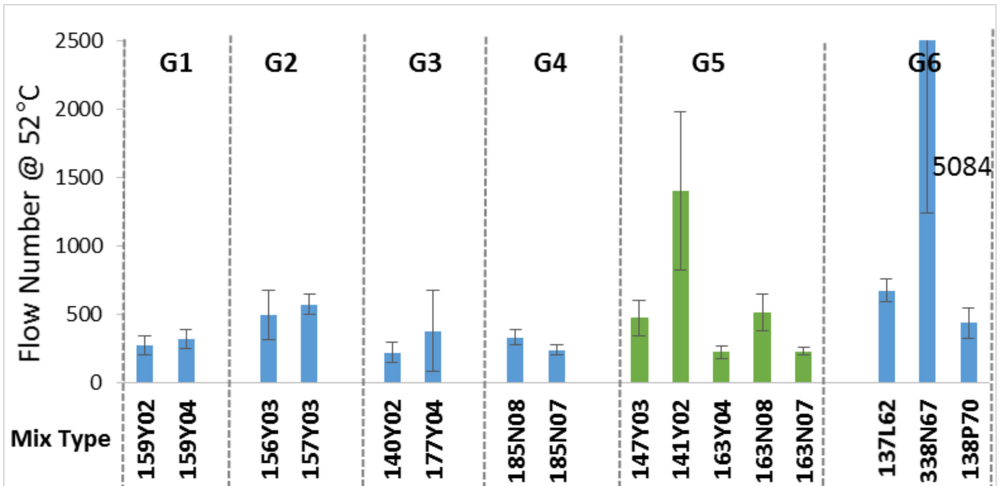
* 1 MPa= 0.145 ksi

Figure 6.9. (a) Dynamic modulus master curves for level binder mixes; (b) phase angle master curves for typical level binder mixes.

6.3.6 Flow Number Test Results

The flow-number tests at 52°C for plant AC mixes are shown in Figure 6.10. In Group G4, the AC mix with lower AC showed a higher flow number. However, the variability of the flow-number test was high, as indicated by the error bar in the figure, overshadowing the effect of mix design parameters on the mixes' resistance to permanent deformation. All mixes had a flow number much higher than 50, which is the minimum number for a traffic level of 3- to 10-million equivalent single-axle loads (ESALS) and indicates that all the tested mixes have an excellent rutting resistance.

The 338N67 mix in Group G6 had an extremely high flow number, which is consistent with the Hamburg wheel-tracking test results that its rut depth was lowest among all mixes because of its highest ABR and RAS content. The mix is very brittle and it experienced higher severity cracks than any other section tested.



Mix Type	159Y02	159Y04	156Y03	157Y03	140Y02	177Y04	185N08	185N07	147Y03	141Y02	163Y04	163N08	163N07	137L62	338N67	138P70
ABR, %	29	29	15	29	30	30	48	48	35	32	29	29	29	60	56	37
RAS, %	0	0	2.5	5	3.1	3.1	5	5	5	5	5	5	5	4.6	5	0
RAP, %	34	34	5	10	20	20	39	39	33	29	24	24	24	51	53	27
AC, %	6	6	5.6	5.7	5.5	6.6	6	6.3	8	7.7	8.1	7.7	8	6.7	6.5	5.8
PG	58-28/58-34		64-22/58-28		58-28/ 58-34		52-34/52-28		70-28					58-28/58-28/58-34		

Figure 6.10. Flow-number test results for plant mix.

6.3.7 Four-Point Bending-Beam Fatigue Test Results

Fatigue cracking, which is a series of interconnected cracks, is one of the major types of distresses in AC. This type of distress is mainly caused by the accumulation of damage resulting from repeated loading lower than the ultimate failure stress. The number of repetitions to failure and strain at the bottom of an AC layer are inversely proportional. The relationship between the two variables can be written as follows:

$$N_f = k_1 \left(\frac{1}{\varepsilon_t}\right)^{k_2} \qquad [6.3]$$

where N_f is the cycles to failure when the initial stiffness is reduced by 50%, ε_t is the applied strain level, and k_1 and k_2 are regression coefficients (Thompson 1987).

Figure 6.11 plots the number of repetitions to failure versus strain levels for four AC mixes. When comparing the mixes 159Y02 and 159Y04, it can be noticed that mix 159Y04 has a better performance; for the same strain, the number of repetitions to failure for mix 159Y04 is higher than for mix 159Y02. By inspecting the composition of these two mixes, it can be observed that the major difference is the PG grade. Mix 159Y04 has a softer binder, which resulted in an improved performance against fatigue cracking. Moreover, the results of mixes 140Y02 and 177Y04 showed that mix 177Y04 is more resistant to fatigue cracking. This improvement in fatigue life performance comes as a result of two main factors: a softer binder and a higher asphalt content.

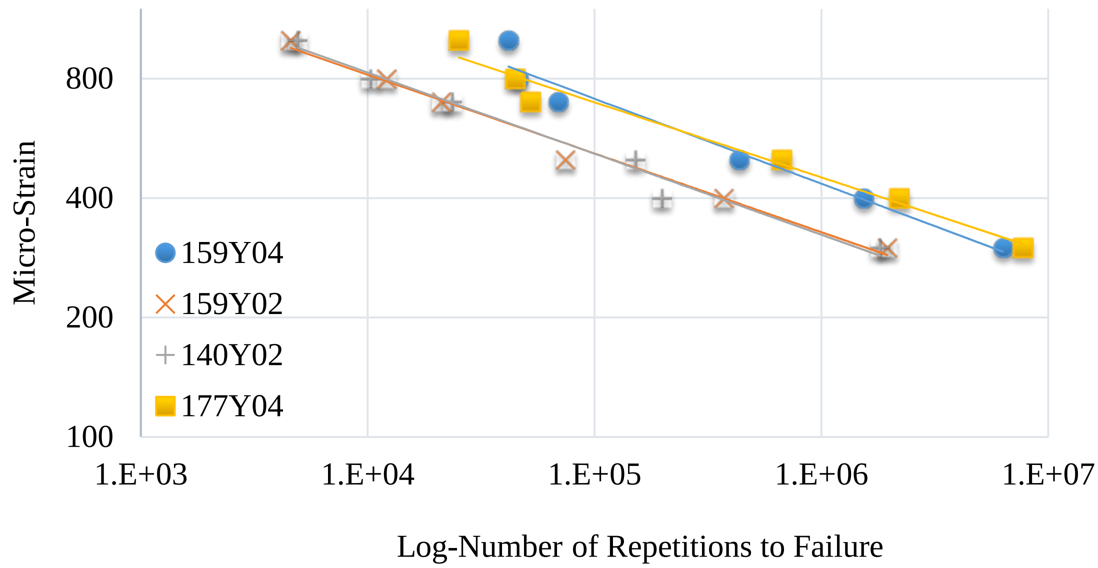


Figure 6.11. Fatigue curves of mixes 159Y02, 159Y04, 140Y02, and 177Y04.

Figure 6.12 shows that 156Y03 and 157Y03 AC mixes behave similarly in fatigue at high strain levels, while at low strain levels 157Y03 performs better. These results show that the effect of RAS and RAP is more pronounced at high strains, while at low strains, the effect of softened binder is the most prominent factor affecting the performance of the mix. The compositions of these two mixes are summarized below.

Figure 6.13 shows relatively similar performance for mixes 185N07 and 185N08 under cyclic loading. Even though mix 185N08 has a softer binder, mix 185N07 has a relatively higher asphalt content. The 0.3% increase in asphalt content balances the effect of having a softer binder, leading to very similar fatigue life performances.

With a lower percentage of RAP and a higher asphalt content, mix 163Y04 exhibited a good fatigue resistance compared to the other AC mixes with the same PG (Figure 6.14). Mixes 141Y02 and 147Y03 showed a relatively similar performance. The effect of the higher percentage RAP in mix 147Y03 was counterbalanced by the increase in asphalt content. The aforementioned mixes also showed a more pronounced effect of the percentage of RAP at high strain.

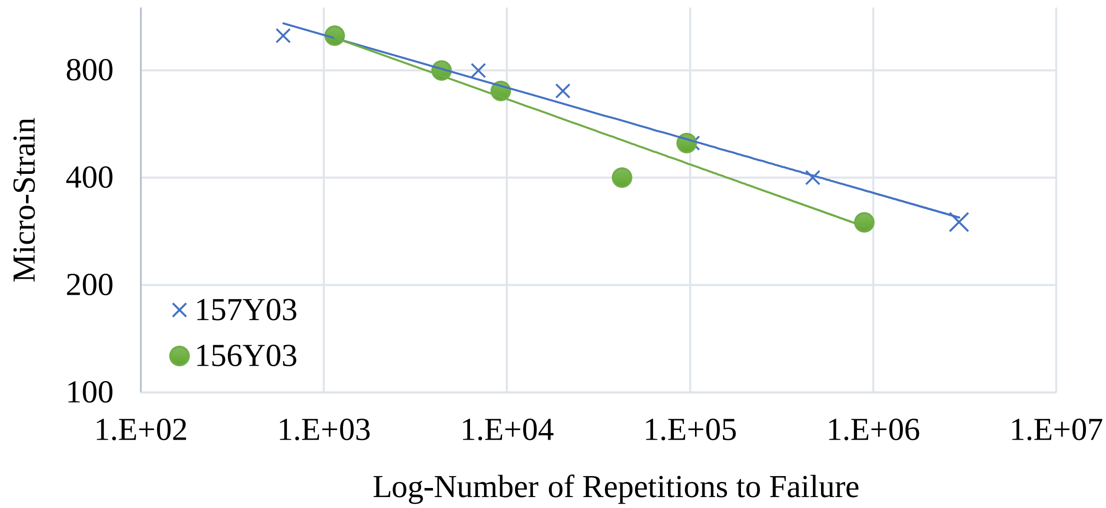


Figure 6.12. Fatigue curves of mixes 156Y03 and 157Y03.

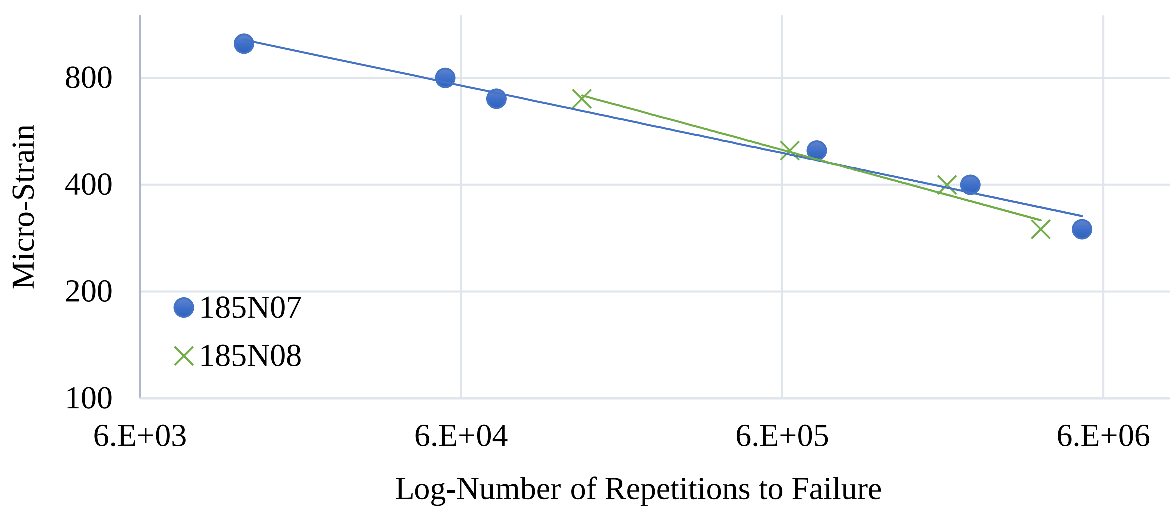


Figure 6.13. Fatigue curves of mixes 185N07 and 185N08.

The slope and intercept of the fatigue curves are plotted in Figure 6.15. The slopes and intercepts of the fatigue curves (K_1 and K_2) were found to be following the same trend for other Illinois mixes (Carpenter et al. 2003). K_2 values range from 4 to 6, which is within the acceptable limits (3 to 6). As expected, log (K_1) values range over several orders of magnitude.

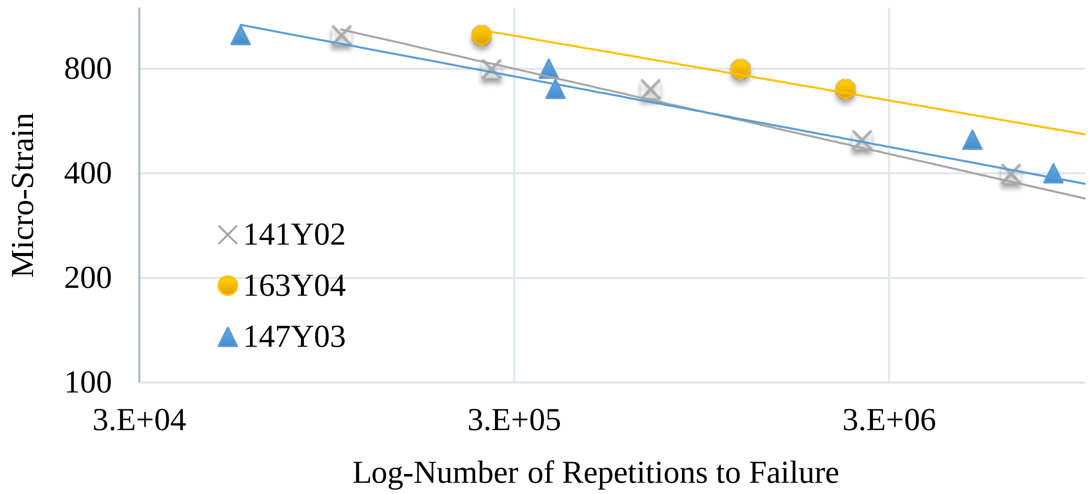


Figure 6.14. Fatigue curves of mixes 141Y02, 163Y04, and 147Y03.

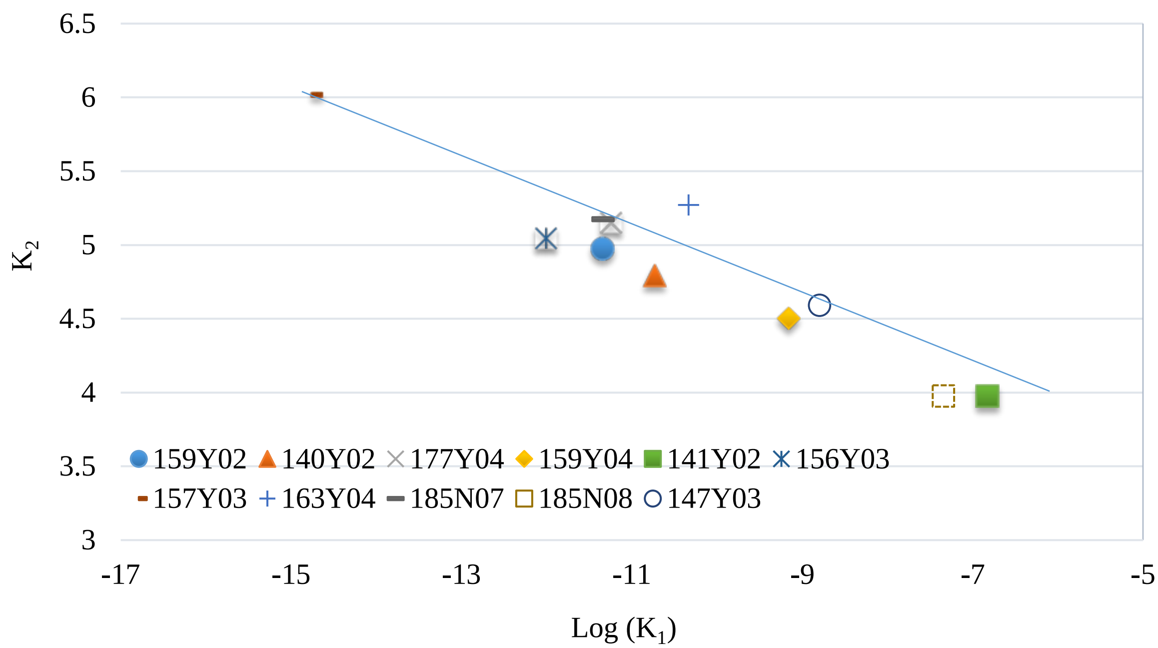


Figure 6.15. Plot of K_2 vs. Log (K_1) for all the tested mixes.

6.3.8 Creep Compliance and IDT Strength Results

Creep compliance was obtained at three different temperatures (-4°F, 14°F, and 32°C) for a period of 1000±2.5 s, using a fixed load to produce a horizontal deformation of 5 to 0.76 mils, using AASHTO T322-07. Creep compliance master curves were obtained using the time–temperature superposition principle at the reference temperature of -4°F, whereas the power-law model was used for curve fitting by using the least sum of squares approach employing the solver function, as given in Equation 6.4.

$$D(t) = D_o + D_1 t^m \quad [6.4]$$

where, D(t) = creep compliance; $D_o, and\ D_1$ are model parameters t = time in seconds, m = slope parameter shows the relaxation potential of AC mixes.

Figure 6.16(a) shows the creep compliance master curves for the level binders. Mixes 163Y04, 163Y07, and 163N08 exhibited similar creep compliance behavior, as these mix designs were similar in terms of binder content, aggregate sources and gradation. However, the other two level binders (mixes 147Y03 and 141Y02) contain relatively more recycled materials and less binder content, which makes them stiffer. Thus, these two mixes were relatively less compliant, especially at -4°F, which suggests that these mixes would experience a higher stress intensity under a low temperature than other mixes and are expected to have a higher rate of crack-propagation speed once a crack is initiated. In general, all the level binders followed the expected trend. It can also be noticed that the creep compliance increased with creep loading time and temperature, which is consistent with findings in the literature.

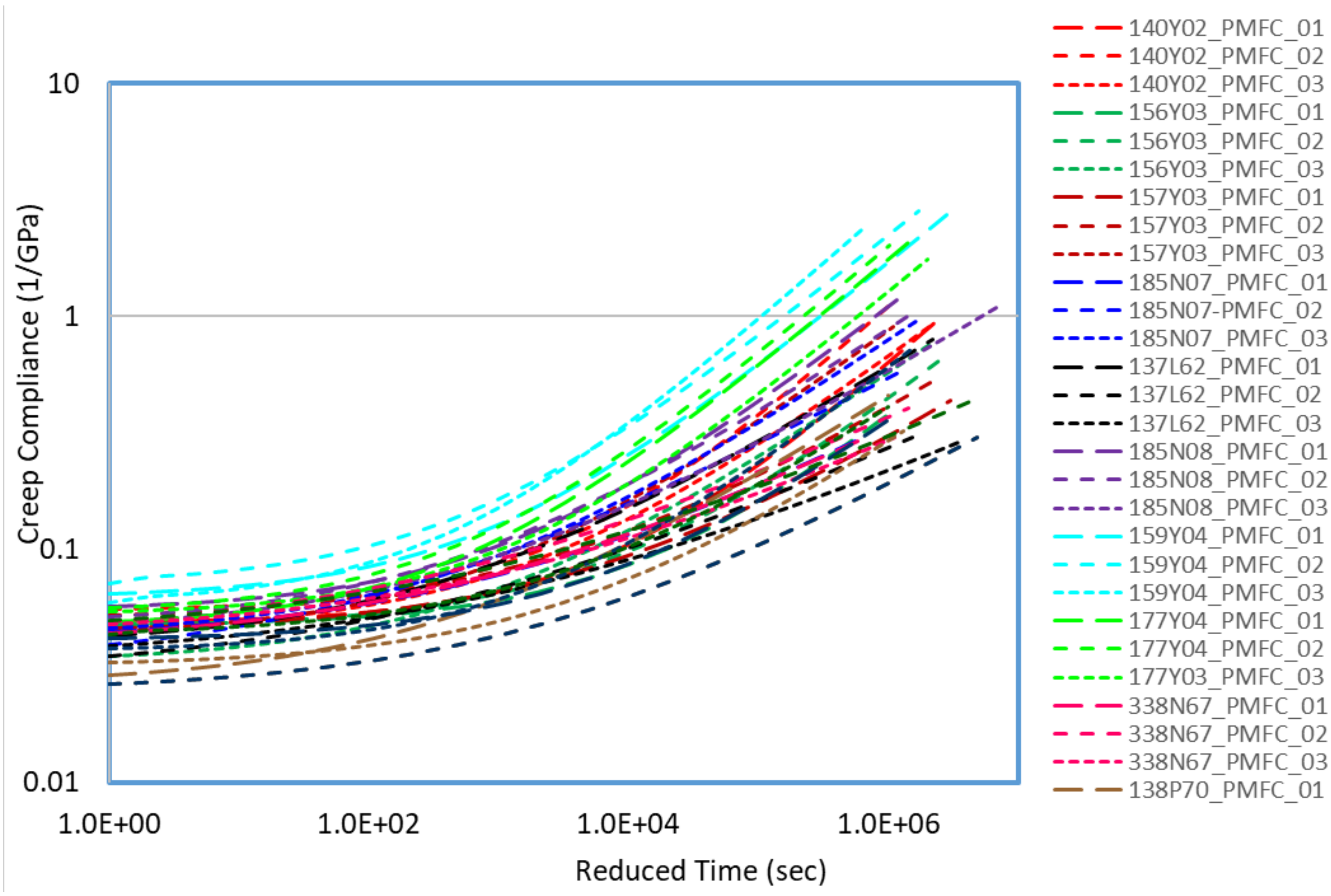
Similarly, Figure 6.16(b) shows the master curve of creep compliances of four surface mixes, where the impact of binder grade and RAP can be observed. As compared to the master curves of level binder mixes shown in Figure 6.16(a), it can easily be noticed that, in general, the difference in creep compliance between these mixes is more apparent. Also, the AC mixes 177Y04 and 159Y04 were both more compliant, as compared with Y02s, because the mix Y02 was produced using a relatively stiffer binder in 2014. Whereas the impact of RAP can be noticed within Y04s, the 177Y04 was relatively more compliant as compared with 159Y04 because the latter contains more RAP. Therefore, Y04 mixes are expected to impede stress intensity in a given cross-section for a given load that will ultimately reduce the crack-propagation speed, compared with Y02. For this reason, the thermal-induced transverse cracks susceptibility in Y04s was lower than that in Y02s.

Figure 6.16(c) shows the impact of binder grade bumping and ABR content. Binder grade bumping is a commonly used strategy to counterbalance the impact of ABR. The creep compliance master curves of both AC mixes were very similar because 156Y03 contains 15% ABR with a binder grade PG 64-22, whereas 157Y03 contains 30% ABR with a relatively softer binder grade, i.e., PG 58-28. Both mixes would be expected to have the same potential to relax the thermal stresses in their corresponding structures.

Figure 6.16(d) shows the master curves of five TRA mixes. The mixes 185N07 and N08 showed more compliant behavior as compared with other TRA mixes because they contain relatively less ABR and softer PG. In general, due to high ABR level and steel slag, TRA mixes are stiffer or less compliant, as compared to other surface AC mixes, as expected.

Three field cores were tested for IDT creep compliance and tensile strength as per AASHTO T322, identified as PMFC_01, PMFC_02, and PMFC_03. Figure 6.17 shows creep compliance master curves of plant-mixed field compacted (PMFC). Generally, no trend was observed in the field cores master curves; however, some cases show an intuitive trend, as third field core compliance values are relatively less and flatter as compared to those of first field cores. This difference could be due to the fact that field aging could induce stiffness and make the mix relatively brittle.

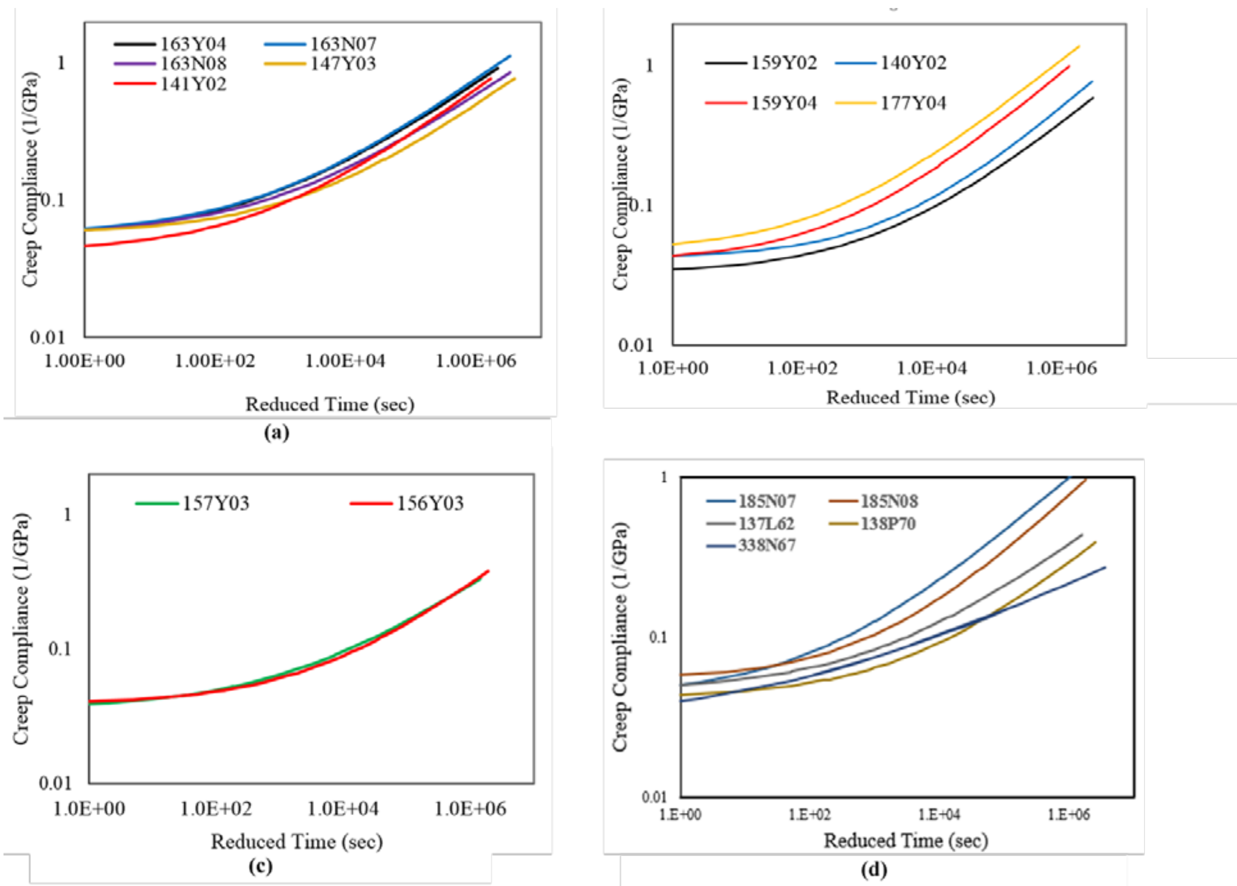
AASHTOWare Pavement ME Design uses the slope or m-value for the thermal-cracking model, which was obtained from fitting the power model using creep compliance master curves. The higher m-value results in better stress-relaxation potential of AC mixes and vice versa. Figure 6.18 shows the m-values, where TRA mixes are relatively lower as compare to other surface mixes, as it contains a higher content of recycled material, which is expected. Also, intuitively, the m-values generally decrease with field aging, which is in agreement with the literature. This could be due to the fact that aged AC mixes become brittle and have relatively less stress-relaxation potential.



*1 GPa= 145.04 ksi

Figure 6.16. Creep compliance master curves of first-round field cores.

Similarly, tensile strength is another important component for the thermal-cracking prediction model of Pavement ME design. The IDT strength is statistically similar across PMLC at 95% confidence interval. Thus, at 14°F (-10°C) all AC mixes show very brittle behavior; and IDT doesn't distinguish the effect of recycled materials. It was shown that testing only at low temperature may not be sufficient to differentiate cracking resistance of AC mixes (Al-Qadi et al. 2015). Hence, there was a need to develop a fracture test at intermediate temperature such as I-FIT. In a few third field cores, a slight increase can be observed as compare to the first one, which might be due to stiffness induced with aging and air voids as shown in the Figure 6.19. However, generally IDT strength is statistically similar across the different field cores, and the effect of aging could not be observed.



*1 GPa= 145.04 ksi

Figure 6.17. Fitted creep compliance master curves of plant mixes: (a) level binders, (b) effect of binder grade and RAP, (c) impact of binder grade and ABR, and (d) TRA mixes.

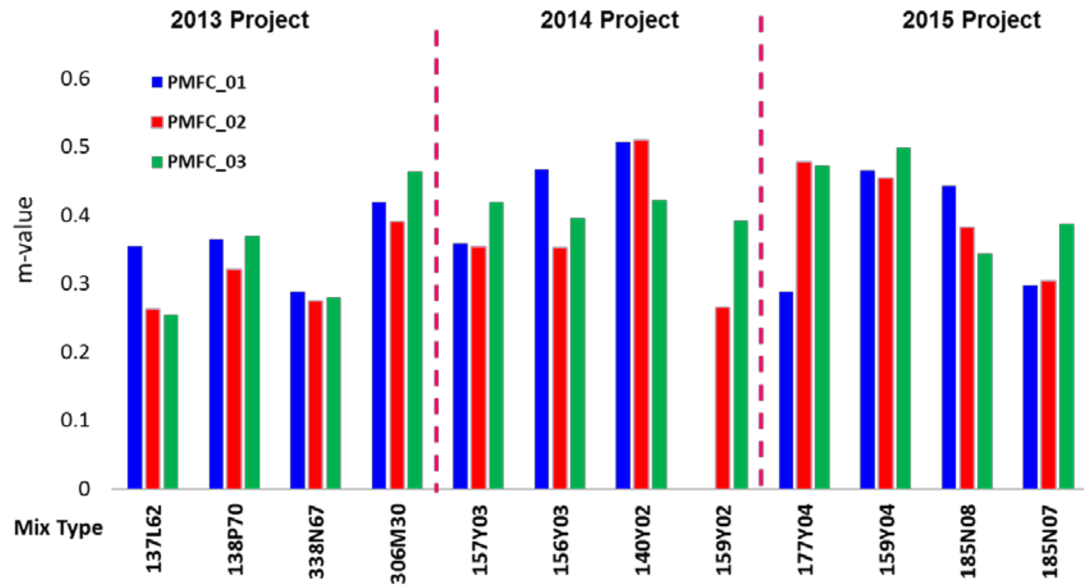


Figure 6.18. m-values from IDT creep compliance master curves.

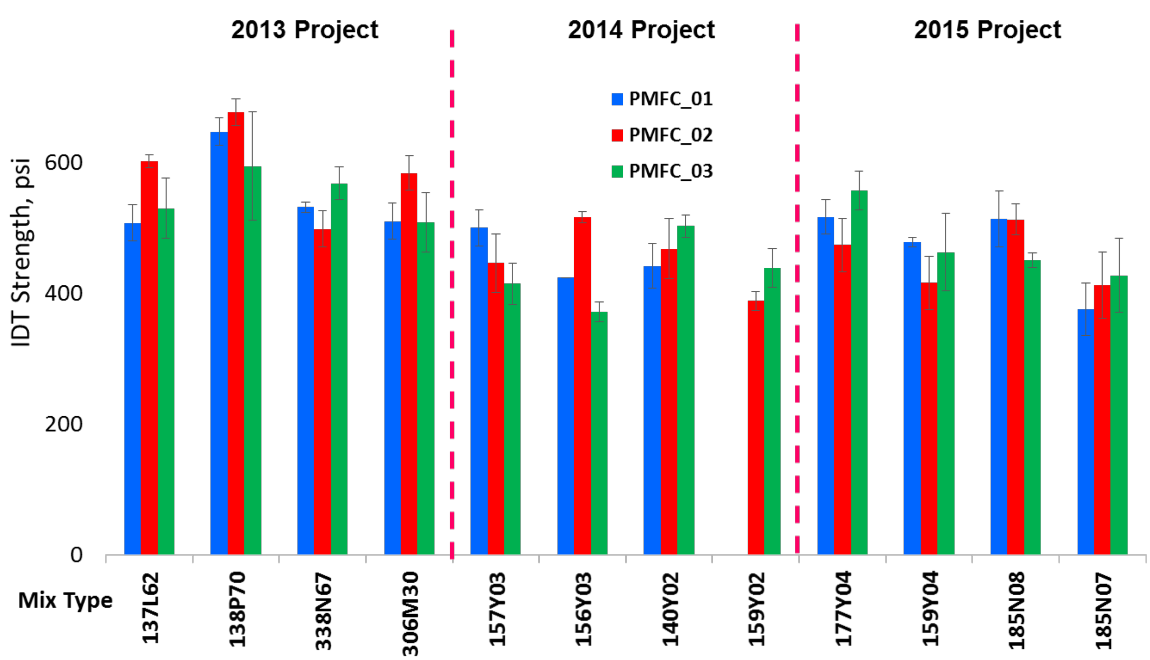


Figure 6.19. IDT strength.

6.3.9 Hamburg Wheel-Tracking Test Results

Hamburg wheel-tracking tests were applied to both plant mix and field cores extracted after construction. Table 6.2 shows the Hamburg test results for the AC-plant mixes and field cores. It should be noted that no level binder cores were evaluated. All of the plant mixes passed the IDOT specification requirement that the average maximum displacement be less than ½ in. The first-round field cores of the 159Y04 mix in contract 60Y04 showed the highest rut depth because the mix used softer asphalt binder and no RAS. It was also noted that the first-round field cores showed more rut depth than the plant mix. In general, the rut depth decreases from the first-round field core to the third field core, due to an increased aging effect and traffic densification.

Table 6.2. Hamburg Wheel-Tracking Test Result Summary

Mix	Neat Binder PG	Designed PG	IDOT Pass Criteria	Average Max Displacement, mm			
				Plant Mix	1st-Round Field Core	2nd-Round Field Core	3rd-Round Field Core
147Y03 (L)	70-28	70-28	15,000	2.8	NA	NA	NA
156Y03	64-22	64-22	7,500	2.0	2.5	2.6	3.5
157Y03	58-28	64-22	7,500	2.5	2.5	3.0	3.8
141Y02 (L)	70-28	70-28	15,000	3.0	NA	NA	NA
140Y02	58-28	64-22	7,500	2.8	5.0	3.6	2.7
159Y02	58-28	64-22	7,500	3.4	3.4	4.9	2.3
185N08	52-34	64-22	7,500	3.7	4.0	3.7	3.8
163N08 (L)	70-28	70-28	15,000	4.4	NA	NA	NA
185N07	58-28	64-22	7,500	4.7	6.0	3.5	2.6
163N07 (L)	70-28	70-28	15,000	3.8	NA	NA	NA
177Y04	58-34	64-22	7,500	4.6	6.7	4.8	3.0
159Y04	58-34	64-22	7,500	4.6	9.9	7.7	6.0
163Y04 (L)	70-28	70-28	15,000	6.5	NA	NA	NA
138P70	52-28	64-22	75,00	3.4	2.3	2.6	2.7
137L62	52-28	64-22	7,500	3.7	4.3	2.7	2.5
338N67	52-28	64-22	7,500	1.6	1.6	2.6	1.6
306M30	58-28	64-22	7,500	NA	2.3	3.2	3.8

Note: L denotes level binder course.

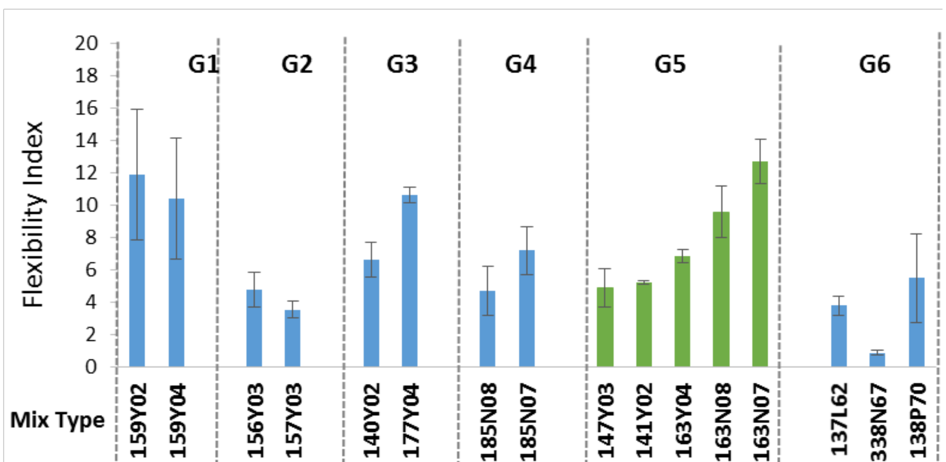
6.3.10 I-FIT Results

The Flexibility Index (FI) obtained from I-FIT using PMLC specimens is shown in Figure 6.20. A higher FI value indicates better cracking resistance. No significant difference was found between two mixes in Group G1. In Group G2, the 157Y03 mix with 29% ABR showed a lower FI value than the 156Y03 with 15% ABR, indicating higher ABR results in a lower FI value despite using a softer binder. However, in general, Group G2 resulted in relatively low FI. The mixes with higher AC in both Groups G3 and G4 exhibited higher FI values, indicating that higher AC content may contribute to better cracking resistance.

For the level binder mixes (Group G5), the 147Y03 and 141Y02 mixes exhibited similar FI values due to similar mix compositions. Mix type 163M in three contracts showed different FI values, possibly due to variation in mix composition that occurred during asphalt-plant production. This difference was also observed in the flow-number test and dynamic modulus test results.

Illinois is currently using a minimum FI of 8.0 for AC surface mixes as part of demonstration projects. The contracts in this study did not have the I-FIT testing specification included, as a result, only three surface mixes (159Y02, 159Y04, and 177Y04) met the current FI requirement. For the level binder mixes (Group G5), it is recommended that an FI value significantly greater than 8.0 be used to control reflective cracking. Hence, an optimized level binder design with RAP and/or RAS should be developed. It is the authors' opinion that the FI should be above 15.0 for a level binder mix if the layer is to provide a crack-retarding function. The addition of RAP and RAS in a level binder course must be reexamined to ensure that the purpose of using a level binder is not jeopardized.

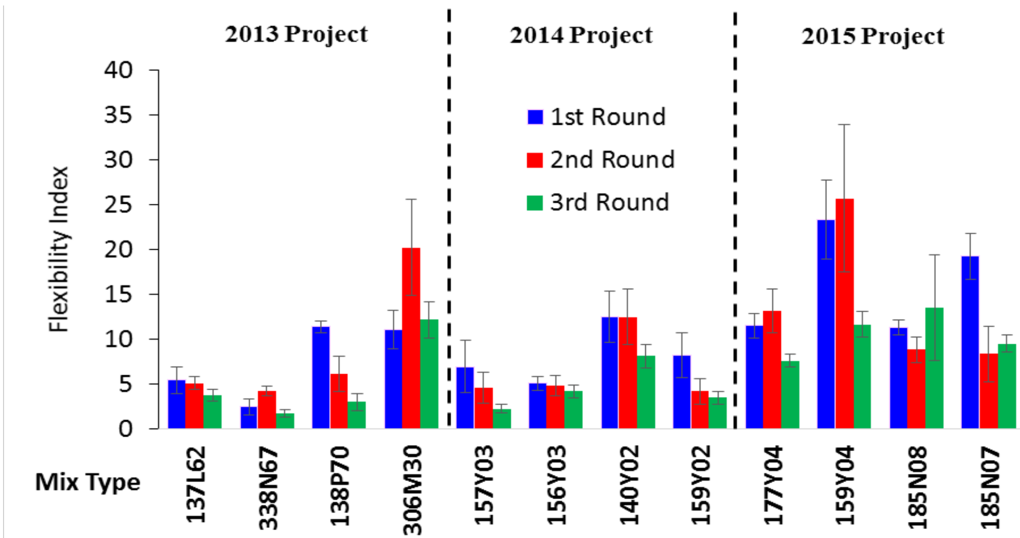
Similarly, Group G6 contains TRA mixes which show relatively lower FI value as expected because it contains high ABR percentage, steel slag and crushed concrete. Among TRAs, 338N67 shows stiffest behavior as it contains 27% recycled concrete aggregates and a relatively higher absorption rate of 1.7%. Thus, I-FIT is a potential cracking test which appears to discern the effect of recycled materials, binder content/type and other mix characteristics as shown in Figure 6.20.



Mix Type	159Y02	159Y04	156Y03	157Y03	140Y02	177Y04	185N08	185N07	147Y03	141Y02	163Y04	163N08	163N07	137L62	338N67	138P70
ABR, %	29	29	15	29	30	30	48	48	35	32	29	29	29	60	56	37
RAS, %	0	0	2.5	5	3.1	3.1	5	5	5	5	5	5	5	4.6	5	0
RAP, %	34	34	5	10	20	20	39	39	33	29	24	24	24	51	53	27
AC, %	6	6	5.6	5.7	5.5	6.6	6	6.3	8	7.7	8.1	7.7	8	6.7	6.5	5.8
PG	58-28/58-34		64-22/58-28		58-28/ 58-34		52-34/52-28		70-28					58-28/58-28/58-34		

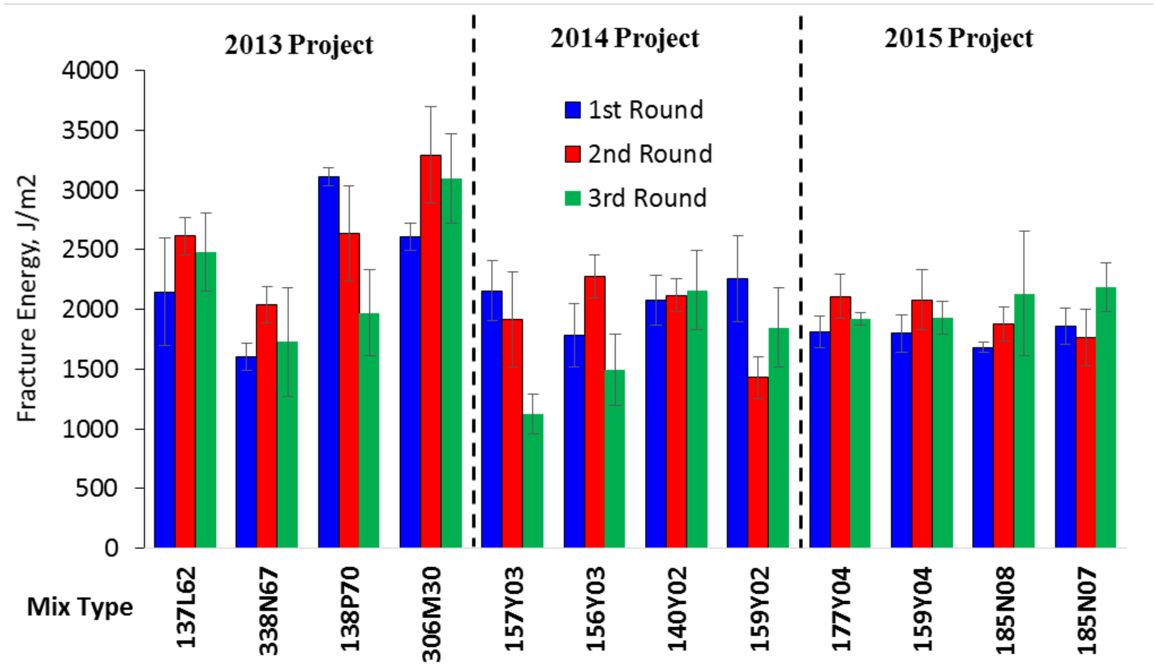
Figure 6.20. I-FIT results for plant AC mixes.

Figure 6.21(a) shows the FI values for third-round field cores. Only surface mixes had field cores. The averaged air void (AV) contents of tested specimens are also shown under each mix type. In general, the FI value decreases from first-round to third-round cores for all AC mixes,



1st AV, %	5.5	3.2	0.2	2.3	4.6	7.6	7.0	8.0	3.3	5.0	5.2	5.1
2nd AV, %	2.1	3.4	0.2	0.7	4.0	4.2	4.1	7.9	2.7	4.5	2.7	2.8
3rd AV, %	1.8	2.2	1.2	2.0	5.2	5.2	4.3	6.4	2.5	4.4	4.6	4.1

(a)



*1 pound force/inch=145.13 Joules/ m^2 (J/m^2)

(b)

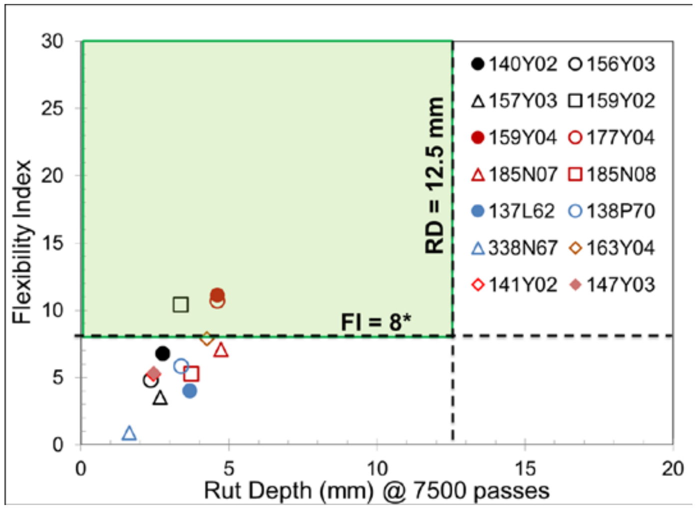
Figure 6.21. Field cores' cracking resistance (a) flexibility index evolution and (b) fracture energy evolution.

indicating that the brittleness of the material increased with time. For the 306M30 mix, the FI of the second-round field core is unexpectedly higher than that of the first-round. As noted, two different RAPs were used for the 306M30 mix; it was thought that the second-round cores sampled pavement sections where segregation had occurred. There was no clear evolution trend for fracture energy, as can be seen in Figure 6.21(b).

The lab results show that Marshall stability is impacted by ABR, asphalt binder content and binder grade. In general increasing ABR and binder grade increase stability, while increased binder content lowers stability. ,. While the level binder has higher stability values as it contains high ABR and polymer-modified binder PG70-28. Similarly, level binder has the lowest Cantabro loss and 156Y03 has the highest loss which might be due to asphalt binder content and binder grade of the AC mix. TSR results provide an indication of moisture susceptibility, however, it might not distinguish between all AC mixes. High COV was observed in the Texas overlay results, and the soft PG, high AC and level binders have higher number of cycles to failure. Thus, OT seems to distinguish qualitatively the AC mixes. The viscoelastic behavior was assessed by dynamic modulus test where E* increases with increase in frequency or decrease in temperature as expected. E* may not easily distinguish all the AC mixes, however high ABR, crushed concrete and steel slag resulted in higher E* and level binders have relatively low modulus. All AC mixes have a flow number higher than 50, which demonstrate excellent rutting resistance because it contains recycled material. Four-point bending-beam results show that softer PG, higher AC, and low RAP/RAS mixes show good fatigue resistance and RAP effect is more pronounced at high strain. Furthermore, creep compliance increases with increase in time, softer PG, higher AC, less ABR and crushed concrete, whereas IDT strength is not very sensitive to these variables and all AC mixes behave very brittle at low temperature and no general trend was found in field cores. Additionally, all AC mixes passed the IDOT rutting requirement of maximum 12.5mm and the rut depth decreased from first to third field cores due to increased aging and traffic densification. Flexibility index increases with higher AC, softer binder, and less ABR. Level binder has relatively higher FI as it is designed to retard cracking. TRAs have relatively lower FI because it contains higher ABR % and other recycled materials. In general, FI decreases from the first-round to the third-round in AC mixes, thus indicating that brittleness of the material increases with time/age. In short, I-FIT appears to be a good tool for evaluating the cracking resistance of different AC mixes.

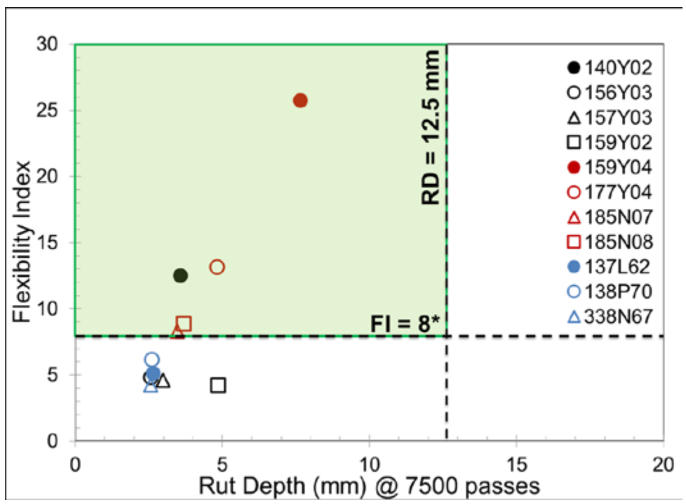
6.4 BALANCED MIX DESIGN APPROACH

An alternative approach to evaluate laboratory performance of AC mixes is the balanced mix design approach. The approach has recently been used by several agencies to complement existing mix design specifications with a specific objective to relate the design to actual performance of the AC mixes. The initial version for the performance-space diagram was used to represent the interaction between the I-FIT and Hamburg test result parameters. The balance diagram for the AC mixes is presented in Figure 6.22 for PMLC and field cores.



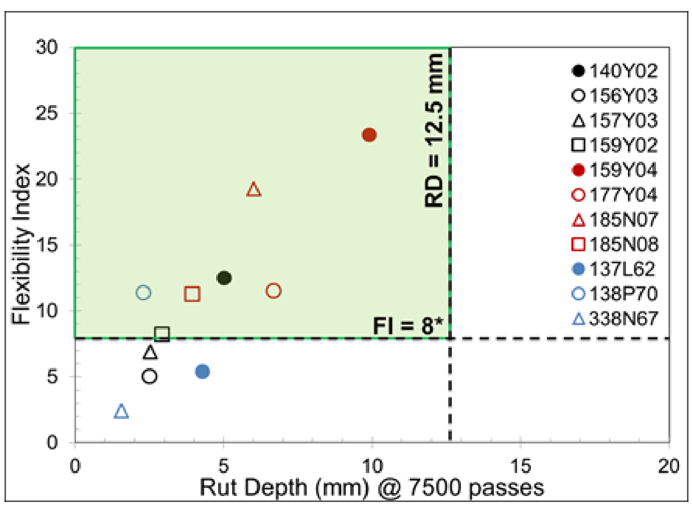
Flexibility Index
Rut Depth (mm) @ 7500 passes
RD = 12.5 mm
FI = 8*
140Y02
156Y03
157Y03
159Y02
159Y04
177Y04
185N07
185N08
137L62
138P70
338N67
163Y04
141Y02
147Y03

. **(a)**



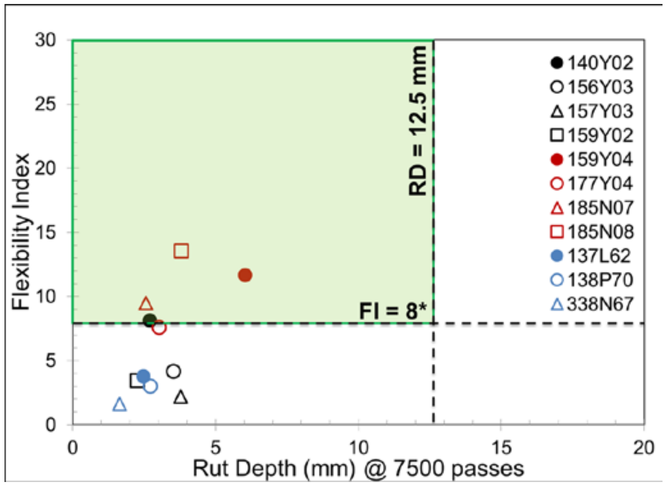
Flexibility Index
Rut Depth (mm) @ 7500 passes
RD = 12.5 mm
FI = 8*
140Y02
156Y03
157Y03
159Y02
159Y04
177Y04
185N07
185N08
137L62
138P70
338N67

(b)



Flexibility Index
Rut Depth (mm) @ 7500 passes
RD = 12.5 mm
FI = 8*
140Y02
156Y03
157Y03
159Y02
159Y04
177Y04
185N07
185N08
137L62
138P70
338N67

(c)



(d)

Figure 6.22. Balanced AC mix for (a) PMLC; (b) Year 1 cores; (c) Year 2 cores; and (d) Year 3 cores.

However, the balance mix design could be better presented in multi-criteria volumetric-performance-space diagrams to evaluate AC mixes' performance in a more holistic approach. An example of such an interaction space is illustrated in Figure 6.23. In the earlier ICT study R27-128 (Al-Qadi et al., 2015), this approach was expanded to consider stiffness of the AC mixes. Adding stiffness to performance-space considerations is important to avoid AC mixes with very high compliance due to various reasons such as very soft binder, open gradation, or high air voids. Even though such AC mixes may exhibit high FI from the I-FIT and pass Hamburg criteria, they may pose performance challenges due to deformation at in-service temperature leading to premature failure. When stiffness is added to the performance diagram, the likelihood of selecting the AC mixes to ensure good performance will be improved.

The stiffness parameter was selected as the secant modulus obtained from the same I-FIT experiment that resulted in the FI values. The secant modulus is calculated as the ratio of peak load to the displacement at the peak load. This parameter was analyzed in this study to compare the modulus of the AC mixes as determined by conventional modulus tests such as complex modulus and/or creep compliance. In this study, the secant modulus was correlated to the modulus, and they were in good agreement. Therefore, it is assumed as a proxy parameter to stiffness. However, it is important to note that the secant modulus is highly specimen geometry-dependent, similar to the FI and Hamburg rut depth.

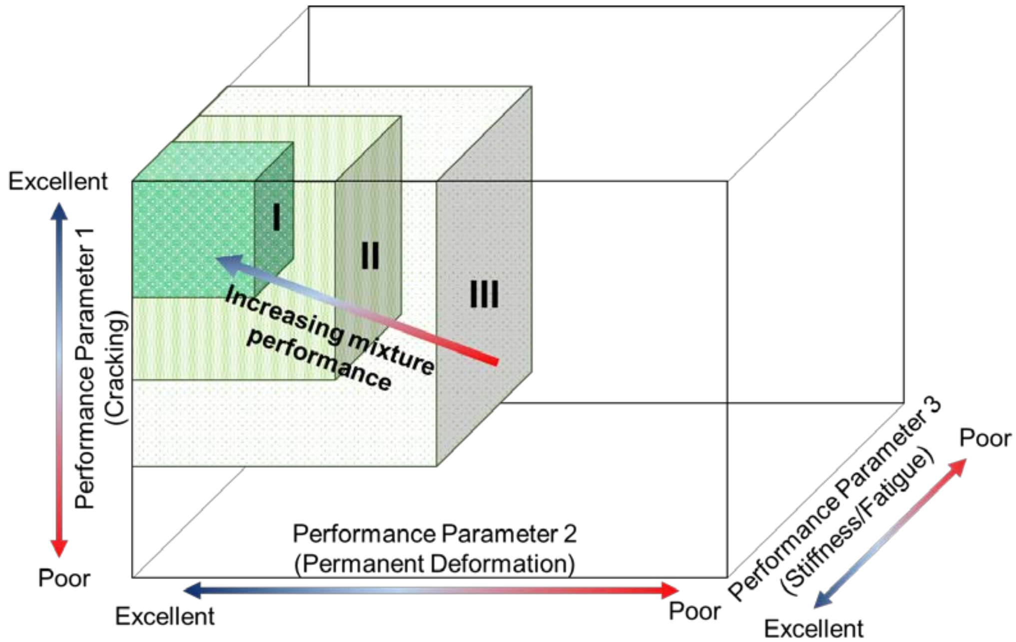


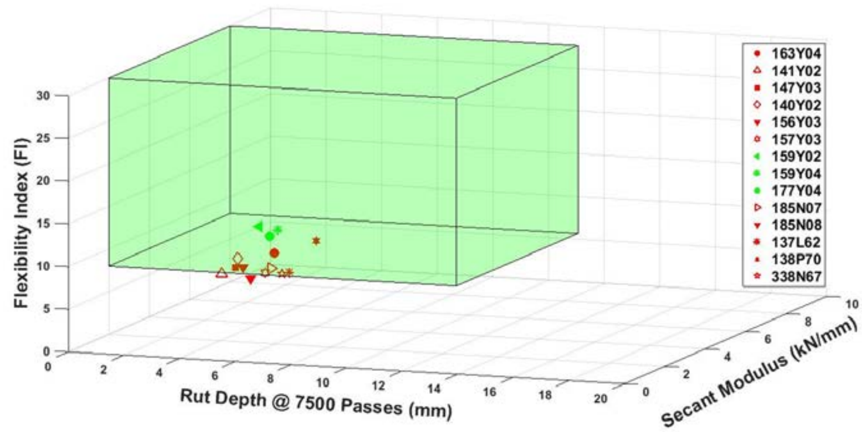
Figure 6.23. Conceptual diagram for the 3-D performance-space.

The AC mixes used in this study were analyzed using the 3-D performance diagram. Results are shown in Figure 6.24, points in green reflect AC mixes passing I-FIT, rut depth, and secant modulus thresholds; points in red reflect AC mixes failing either I-FIT or rut depth thresholds; while points in yellow reflect AC mixes passing I-FIT and rut depth, but failing secant modulus threshold. Secant modulus is assumed acceptable between 11.4-57.2 kip/in (2-10 kN/mm). The range of secant modulus for the AC mixes shown in Figure 6.24 is 20.0-47.5 kip/in (3.5-8.3 kN/mm). The AC mix with highest secant modulus (47.5 kip/in) is the 338N67 mix, which also resulted in the lowest FI (0.91). The AC mix with the lowest secant modulus (20.0 kip/in) is 185N07 mix, which resulted in an FI, value of 7.1. The AC mixes with secant modulus 20.0 to 28.6 kip/in (3.5 to 5.0 kN/mm) resulted in relatively higher FI values. There was a clear trend as stiffness increases, FI decreases. The AC mixes in this study are, in general, stiff mixes due to high recycled material content. Therefore, stiffness parameter may not be the controlling criterion herein.

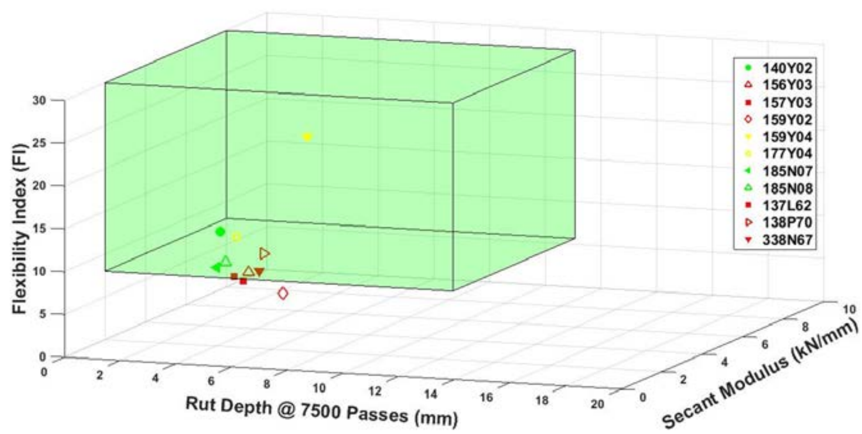
The following approach is recommended in the implementation and interpretation of the multi-criteria performance analysis:

- The AC mix stiffness is critical for pavement performance; therefore, a minimum stiffness criterion is required. This will ensure the rejection of borderline AC mixes which sometime may not be captured by Hamburg WT criterion. The threshold can be initially determined by a comprehensive analysis of I-FIT results of various AC mixes.
- Once the AC mix passes the stiffness criterion, a balance mix considering FI and rut data can be plotted to select the AC mix with optimal cracking and rutting resistance.

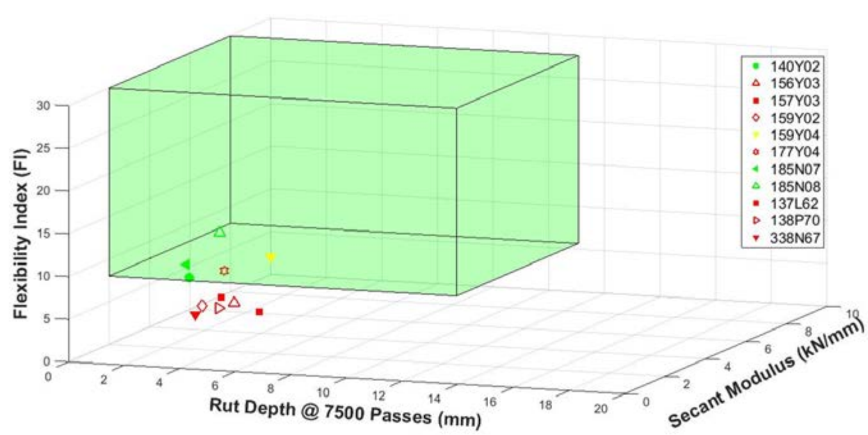
- Alternatively, a multi-performance scoring criterion can be developed to calculate an overall weighted performance score of a mix for a given application (e.g., 0.4*FI + 0.3*Rut + 0.3*Stiffness for AC surface mixes and 0.5*FI + 0.25*Rut + 0.25*Stiffness for a leveling binder mix where flexibility is more important).



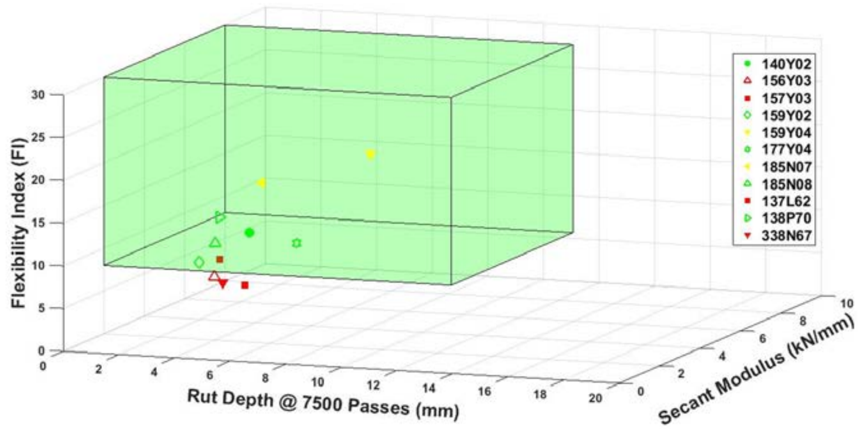
(a)



(b)



(c)



(d)

*1 kN/mm = 5.7 kip/in

Figure 6.24. 3-D performance diagram for (a) PMLC; (b) Year 1 cores; (c) Year 2 cores; and (d) Year 3 cores.

CHAPTER 7. PAVEMENT PERFORMANCE MONITORING

Distress surveys were conducted prior to construction, post-construction, and each spring thereafter for all projects. In addition, profile data collection was conducted on a similar time interval as distress surveys. From pavement distress conditions and profile data, a condition rating survey (CRS) value can be determined (IDOT 2010, 2014). The CRS value is IDOT's main rating and communication method for determining pavement condition and program-inventory condition. This chapter presents the final dataset collected and their overall trends.

7.1 DISTRESS SURVEYS

Distress survey data were collected on the sections using established distress criteria (IDOT 2012a). The datasets consist of pre-construction, post-construction, and springtime surveys of 2014, 2015, 2016, and 2017. Summaries of the distress surveys by project, segment, and date are presented in Appendix B. To present data trends, the data summaries were normalized to standard units and presented by stacked-bar charts for each distress type, which are also presented in Appendix B.

7.1.1 2013 Let Projects

Part of the annual distress survey is to take photos at similar locations, with each survey providing a visual progression of distress with time. Typical photos representing each section from the 2013 letting are presented in Appendix E. Based on data collected to date, the following summary comments are offered for each project:

For the 26th Street project, the first winter of 2013–2014 resulted in significant amounts of high-severity centerline distress. The distress was of such severity that in late 2014 approximately 20% of the centerline joint length was removed and repaired with a narrow longitudinal patch. When the 2016 and 2017 surveys were performed, it was evident that the remaining centerline joint continued to degrade. Where the centerline joint was repaired by longitudinal patching, this repair itself is having performance problems. Much of the repair was rated in a high-severity condition, as a result of the patch repair showing signs of cracking and raveling of the patch.

Transverse cracking on 26th Street has grown each year, with the largest increase occurring after the second winter. Approximately 70% of the transverse cracking length prior to rehabilitation had reflected through the surface. Of those cracks, approximately 50% are medium- to high-severity distress levels caused by the width of the crack more so than deterioration of the crack. In 2017, those cracks already rated as high-severity due to the width of the crack were starting to exhibit secondary cracking of the AC adjacent to the original wide cracks.

For the Harrison Street project, after the winter of 2013–2014, little distress was noted other than transverse cracking from underlying joints and cracks. For this report, Harrison Street data were separated into the composite section from Station 3+00 to 54+00 and the full-depth AC section from Station 54+00 to 60+00. By 2017, the level of transverse cracking on the composite section was 103% of the original length. In the same period, transverse cracking on the full-depth AC segment increased well over 100%. However, upon review of the pre-construction survey, it was noted that a relatively new repair had been made in approximately 100 ft of the 600-ft full-depth AC segment. For this reason, the reflective cracking measure does not produce meaningful results, due to cracking in this area being new cracks and not reflective. Therefore, the reflective cracking percent data have been removed from the analysis. In the full-depth AC section, approximately two-thirds of the transverse cracking is medium or high-severity. For the composite section, nearly all the cracking is at a medium- or high-severity level due to the width of the cracks and not the extent of the crack.

Also of note on Harrison Street is the amount of alligator or fatigue cracking that has exceeded the pre-overlay amount by approximately tenfold in both the full-depth AC and composite sections. Alligator or fatigue cracking is a reflection of the structural support of the road or underlying materials' performance. The surface course was being lost in some areas. A site visit indicated that apparently the alligator cracking is being caused by heavy use of the adjacent aggregate shoulder for parking. Commercial trucks driving on and off the shoulder to park and then backing into a loading dock is another cause of the fatigue cracking distress.

Although at a low severity, Harrison is the only project to show block cracking just two years after construction.

The Richards Street project showed fatigue cracking on over 6% of the roadway length after the winter of 2015-2016. By spring of 2017, the measure was approximately 9% of the pavement length. Without a direct investigation, it is not clear if a part of the cracking is due to structural issues or debonding at the surface course/level binder interface. In 2015, other distresses such as raveling/weathering/segregation and longitudinal cracking, which are closely related to the properties of the surface material, began to appear and were relatively unchanged in 2016 to 2017. In comparing the three TRA sections constructed in 2013 Richards Street is the best performing in respect to transverse cracking; but the amount of fatigue cracking is a great concern, as many of these areas will likely require full-depth repairs in the near future.

Wolf Road was selected as a comparison project due to its location next to the Harrison project. See Table 2.1 for key mix details. After four winters, the rate of cracking has slowed. Transverse cracking increased just 2%, from 46% in 2016 to 48% in 2017. Centerline distress increased slightly in extent and severity. Raveling and weathering distress were first noted in 2016 at a low severity, three years after construction, which is relatively unchanged for 2017. The severity and extent of these distresses are less than the 2013 let TRA projects. Wolf Road continues to perform markedly better than the TRA sections. It should be noted that Wolf Road was extensively patched prior to the letting of this project. Due to extensive patching, the resulting joint spacing is approximately 11.2 ft. When joints are spaced this close together, it is not uncommon for pavement joints to basically "lock up" and act as a hinge. In such cases, movement occurs at every other joint or perhaps every third joint. This low joint movement may explain the low amount of reflective cracking from the joints. On a relative level, after four winters, Wolf Road contains distresses similar to or less severe than the TRA sections after the first or second winter, depending on type of distress compared.

After the fourth winter on the projects constructed in 2013, it is clear that performance of the standard mix used on Wolf Road is providing superior performance to that of sections built with TRA mixes.

7.1.2 2014 Let Projects

Of note in reviewing the data is that, overall, there is much less distress on the 2014 let projects than is seen on the 2013 let projects at the same age, especially with respect to transverse cracking, centerline joint performance, and raveling/segregation. The main distress for the 2014 let projects was transverse cracking, yet on some projects limited transverse cracking has developed and at lower severities. Of special note in reviewing the data is that all of US 52 and Segment 2 of Washington Street have much thicker AC, with 3.75 in of AC after milling left in place to build the new AC overlay upon. This observation resulted in “thick” and “thin” designations being added to each section for analysis.

Although the 2014 projects were established to evaluate various mixes, there is a strong correlation between reflective cracking and pavement family. It should be noted that, while detailed, the data in this study are limited. It is not known if these early differences will be maintained with time. The 2014 let projects of Crawford/Pulaski and Segment 1 of Washington Street were overlays on bare-PCC pavement, or the milling operation removed the pre-existing AC down to the underlying PCC pavement. These were all given a “thin” designation. US 52 and Segment 2 of Washington Street were given a “thick” designation.

The main feature of Crawford Avenue/Pulaski Road was a head-to-head comparison of AC mixes utilizing 15% and 30% ABR. The higher ABR used a softer PG58-28 binder to replace the PG64-22, which followed grade-reduction specifications in place at the time. This practice was intended to result in similarly performing mixes. Transverse cracking was evident after the first winter, with just under 20% of pre-rehabilitation joints and cracks reflecting through the new overlay. After the second winter, crack reflection was nearly 50%, and after three winters approximately 75%. The transverse cracking is also increasing in severity each year. Even though different AC mixes were used by direction, data thus far do not indicate that one mix or the other is resulting in more or less cracking or markedly different severity levels. Cracking and distress overall are similar in each direction. Even though the project was broken into segments for monitoring and data analysis, there does not seem to be any major performance difference between the segments—although Segment 1 demonstrated less reflected transverse cracking, at slightly lower severity levels.

The main feature of the surfaces on US 52 (IL 53 to Laraway Road—Contract 60Y02) was similar AC mixes with respect to ABR and PG grade, with one mix using both RAP and RAS while in the opposing direction the mix contained only RAP. The cross-section is grouped in the “thick” AC family, with 6 in of AC in place after milling, upon which the new overlay was placed. After the first winter, there was less than 5% reflective cracking on this project. After three winters, the rate of reflective cracking is 25%, of which the vast majority remains at a low severity. The AC mix with RAP only resulted in slightly less reflective cracking. After two years, nearly all of the project had experienced low severity raveling/weathering distress, which was similar for both mixes. Likewise, centerline-cracking distress began to appear at a low severity throughout the project after the second winter.

The main feature of US 52 (Laraway Road to Gougar—Contract 60N08) was a TRA mix that used a very soft PG52-34 asphalt binder to counter a 48% ABR. Like all of US 52 projects, the cross-section is in the “thick” pavement family, with 6 in of AC in place after milling, upon which the new overlay was placed. After the first winter, less than 1% of the cracking reflected through the overlay. After the second winter, 5% of the pre-overlay transverse cracking reflected through the overlay, with the majority being of low severity and only a few cracks being rated at a medium severity. Other distresses were nearly nonexistent.

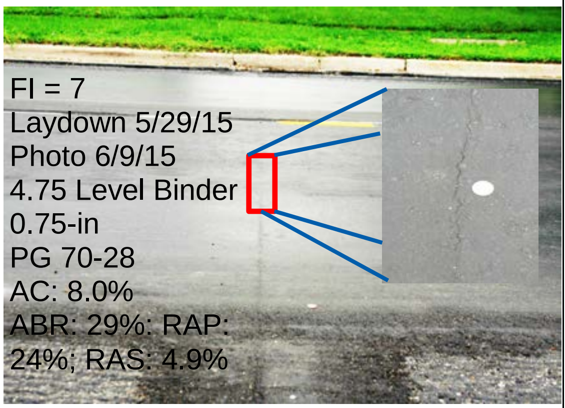
The main feature of US 52 (Gougar to Second Street—Contract 60N07) was the use of the same TRA mix-gradation and source materials as in contract 60N08; however, a slightly stiffer PG58-28 asphalt binder was used. The cross-section is in the “thick” pavement family, with 6 in of AC in place after milling, upon which the new overlay was placed. After the first winter, less than 1% of the cracking reflected through the overlay. After the second winter, 10% of the pre-overlay transverse cracking reflected through the overlay, with nearly all being of low severity. Only three linear feet each of medium- and high-severity transverse cracking were recorded. The only other distress was some limited longitudinal cracking.

The Washington Street (Briggs Street to US 30) project featured the use of similar AC mixes at the 30% ABR level—one mix with RAP and RAS, and one mix using only RAP to obtain the desired ABR. Both mixes used a PG58-34 asphalt binder. There are two different cross-sectional segments on this project that have stark performance differences. Segment 1 was a bare-PCC pavement at the time of rehabilitation. After the first winter, 24% of cracking was found to have reflected through the new overlay of Segment 1 and just over 30% after the second winter. Section 2 was in the thick-pavement family, with an existing AC layer of 3.75 in remaining in place prior to the new overlay. This segment contained just over 1% reflective cracking after one winter and just under 3% after two winters. No other distress was recorded on the segment. Thus far, there was not a significant difference in performance by direction/mix on the project.

7.1.3 Transverse Cracking of All Sections

In reviewing the broader dataset, major performance differences after the first and second winter were observed based on the type of pavement family. Projects in the thin family contained more cracking than those in the thick family for a given age of overlay. The only exception is the 2013 TRA project on 26th Street, which did not seem to follow this trend. Although being in the thick-pavement family, it is performing in a similar fashion as the thin-pavement family.

Direct observations of transverse cracking distress in newly placed level binder is an indication of how challenging preventing reflective cracking on overlays of jointed concrete pavement can be. Figure 7.1 presents an observation photo of reflective cracking in Segment 1 of Washington Street. This segment was bare-PCC pavement that a level binder mix was placed. After 11 days, each joint had reflected through the level binder to some degree. The level binder mix provided an FI of approximately 7.0. The East bound surface FI was 10.2 while the West bound FI was 6.9. After two winters both surface mixes were cracked at this location. A formal survey was not conducted, but it was observed that very few transverse cracks reflected through the level binder on Segment 2 (thick family) of Washington Street. Subsequent crack surveys reinforced this observation. Projects on US 52, which also left a substantial layer of AC in place (approximately 6 in) also exhibited limited transverse cracking after the first winter.



(a) Reflective crack in level binder

(b) Reflective crack in surface

Figure 7.1. Washington Street, Segment 1 progression of crack on same joint through "thin" overlay.

7.2 PAVEMENT-PROFILE MONITORING

The final pavement-profile dataset is presented in Appendix C, by segments, for each project. Previous reports presented data as collected for pre- and post-construction, winter and spring (Lippert et al. 2015, 2016).

The first data run on the 2013 let projects was undertaken in late fall of 2015 (after the second winter) with follow-up data collection in the spring of 2016 and 2017 (after the third and fourth winters). To fill in some of the early data points, IDOT provided data from their pavement-management system. The IDOT data was averaged by tenths of a mile and reviewed and screened for intersection/traffic influences, as described in chapter 5, to develop project averages.

As noted in Chapter 5, the initial pavement smoothness on these projects was higher than what might be expected for new pavement. Over time, the pavements increased in roughness at a similar rate. The resulting IRI trends are presented in Appendix C.

Although, rutting data was not collected post-construction or after the first winter, it can be assumed that rutting was limited to a few hundredths of an inch from early traffic. Of the 2013 projects, Richards Street and Harrison show the most aggressive rutting increase over time.

For the 2014 let projects, the work plan called for data collection post-construction, winter-frozen conditions, and each year in the spring after complete thawing. The final dataset is presented in Appendix C. Previous reports presented data as collected (Lippert et al. 2015, 2016).

A review of the IRI data shows the type of mix placed as part of this experiment did not result in significant differences. The final data show greater differences from project to project overall than by different AC mixes when placed in opposite directions. Winter conditions had limited impacts to pavement smoothness, with urban cross sections showing little to no impact from being frozen. Rural segments showed only slight change with freezing. After winter, the pavement impacted by frozen conditions returned nearly to conditions found the previous spring.

Appendix C presents the IRI and rut depth history of the 2014 let projects. Each section is gradually increasing in rutting. Rutting is still in the early stages of observation. More data over time will be needed to show differences in performance of the various mixes and segments.

7.3 CONDITION RATING SURVEY (CRS)

Since the mid-1970s, IDOT has used the CRS Program to determine the condition of the pavement inventory, evaluate rehabilitation alternatives, and determine improvement needs. At the core of this subjective evaluation is a range of values from 1.0 (nearly impassable) to 9.0 (new or ideal condition) to rate a pavement section (IDOT 2010, 2014).

Key factors are needed to determine CRS values, which are IRI and rutting from profile testing and a visual review to determine the five most predominate distresses, including their extent and severity. With this information, along with the pavement family type (overlaid jointed-PCC pavement as an example in this study), a CRS value can be calculated.

For pavements in this study, CRS values translate into the following performance groupings:

Poor (1.0 ≤ CRS ≤ 4.5). The pavement is critically deficient and in need of immediate improvement.

Fair (4.6 ≤ CRS ≤ 6.0). The pavement is approaching a condition that will likely necessitate a major improvement over the short term.

Satisfactory (6.1 ≤ CRS ≤ 7.5). The pavement is in an acceptable condition (low end) to a good condition (high end) and is not in need of a major improvement, but pavement preservation treatments may be applied.

Excellent (7.6 ≤ CRS ≤ 9.0). The pavement is in excellent condition.

For the projects let in 2013, each project for which data was available had a CRS determined. The result was that the Harrison Street full-depth AC project segment had the lowest CRS, with a value of 4.3 after four years of service. However, note that this is the only full-depth AC segment. All other pavement segments are composite pavements, the lowest composite pavement CRS was the 26th Street Project with a CRS value of 5.0. The centerline distress contributed to the low CRS value on 26th Street even though it is in the “thick” pavement family. However, this mix has high ABR, including 4.6% RAS). On the other hand, Wolf Road had the highest CRS value of this letting group with a 6.6. The mix had the lowest ABR and no RAS.

For the projects let in 2014, Crawford/Pulaski had the lowest CRS, with values ranged from 6 to 7. The highest CRS value of this letting group, 8.4, was determined for both Segment 2 of Washington Street and US 52 (Laraway to Gougar – 60N08), a TRA section. These high values were after just two years of service. It is also worth noting that the highest CRS values were pavements in the thick-pavement family and had soft binder that increased in content by 0.5-0.8. Results of the CRS review are given in Appendix F.

CHAPTER 8: ANALYSIS

The purpose of this chapter is to present relationships between AC mix properties and pavement performance. The relationships explored were limited to those IDOT would most likely adopt.

8.1 PAVEMENT-CRACKING ANALYSIS

As seen in Table 2.2, a variety of tests was performed on the AC mixes to gain as much information as possible on driving factors impacting pavement performance. While informational, some of these test do not lend themselves to timely mix evaluation, selection, and approval in a contractor or agency mix-lab environment. For this reason, only those tests that could be implemented as part of a robust AC mix-design phase were considered for detailed analysis. Table 2.2 also shows that several of the tests were not available for the 2013 let projects.

8.1.1 Selection of Mix-Test Results for Analysis

The tests reviewed for analysis were Cantabro loss, I-FIT, Texas overlay, Hamburg wheel-tracking, tensile strength, and Marshall stability. All of the tests can be conducted currently by IDOT's districts, contractors, or at least its central laboratory. A review of the AC mix data in Appendix B shows the tests were further reduced for consideration after exploring potential correlations to transverse cracking distress that could produce meaningful results. Only three projects are available for analysis from the 2013 let project group because the comparison project on Wolf Road did not have the plant mix taken for testing, resulting in the lack of FI and Texas overlay data. The mix tests presented are FI from the I-FIT test and the Texas overlay test cycles to failure.

8.1.2 Selection of Representative Pavement Performance Measure for Analysis

Due to the various letting and construction times, the ages of the pavements at the end of this study varied from two to four years. For this reason, transverse cracking data for all projects under study are available for only the first two years of pavement life. With each additional year of performance, the number of projects providing data is reduced.

The simplest performance data element reviewed was cracking rate. The rate was determined by normalizing the amount of cracking in 1,000 ln-ft of pavement. All levels of cracking were totaled for each segment for which survey data was collected. These data are also presented in Appendix B in bar chart form. Transverse cracking is independent of contractor workmanship and is more dependent on the pavement cross-section, traffic loading, and AC surface-mix properties.

Other distresses in the dataset do not lend themselves to such detailed analysis because they are not present on all projects and, in many cases, are binary in nature (once measured, the distress is recorded on nearly the entire project (i.e. raveling/weathering/segregation and block cracking distress). Alligator/fatigue cracking has the issue of being more closely related to loading or underlying support issues, which are unrelated to AC surface-mix properties. Centerline distress could be partly attributed to contractor workmanship. For these reasons, transverse cracking rate was used as the main performance indicator.

8.1.3 Analysis of FI, Texas Overlay Test Versus Transverse Cracking

The resulting matrixes used for analysis are presented in Appendix G. Analysis was conducted on the dataset as a whole for FI and Texas overlay cycles, Figures 8.1 and 8.2, respectively. The I-FIT FI produced results consistent with expectations, in that the higher the FI the less transverse cracking in the data. The data show that the extent of transverse cracking in projects is closely related to the pavement family for the projects in this study. Then within either the thin- or thick-pavement family, the resulting AC FI values impact the amount of transverse cracking. The relationship is seen to be consistent for each year's dataset for all the data and for the thick and thin subgroups.

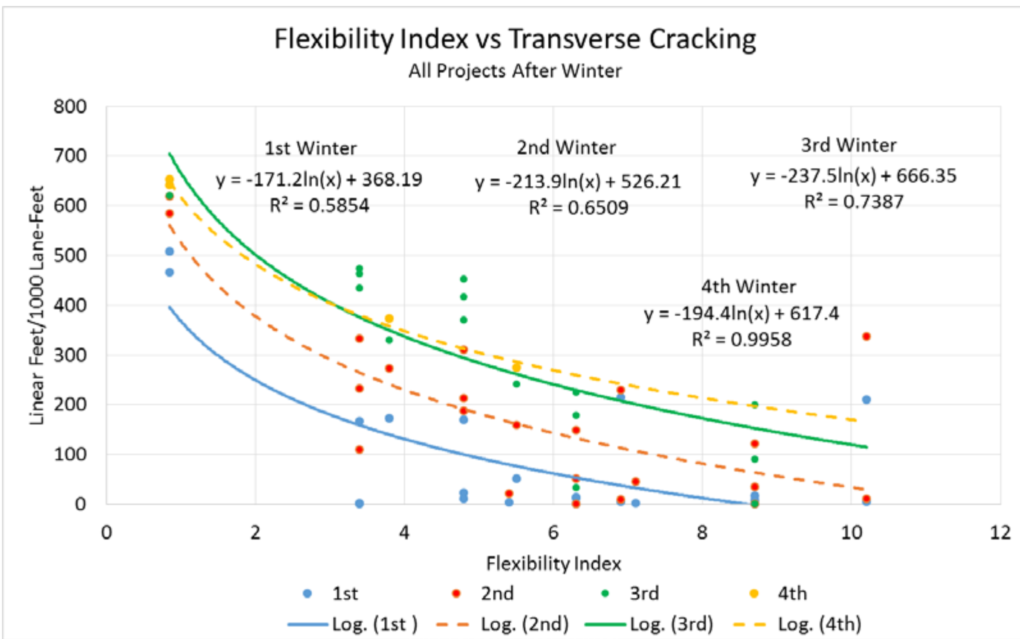


Figure 8.1. Flexibility Index (FI) relationship to transverse cracking on all projects.

The use of FI values verse transverse cracking produces reasonable relationships that indicate AC mixtures can be controlled to reduce cracking. The slope and trends of the relationship indications that with respect to surface mix, FI values in the 8 to 10 range provide the greatest benefit in reducing transverse cracking.

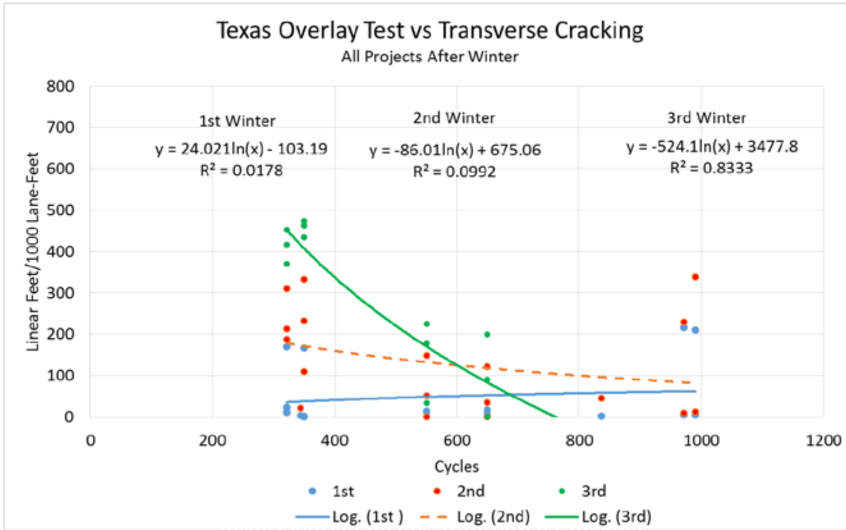


Figure 8.2. Texas overlay test cycles relationship to transverse cracking on all projects.

8.2 CONDITION RATING SURVEY (CRS) ANALYSIS

The CRS value is a subjective measure that summarizes all aspects of pavement condition (distress, ride and rutting) into a single rating number. For this analysis, a similar approach was used as cracking above using CRS values with time and FI values, such that no meaningful relationship can be determined the first year. This is due to the boundary condition of 9.0 for new pavement in the rating system. After the first year, CRS values start to change due to the aging and related distress development.

Relationships were explored similar to cracking above for all pavement families, thin families and thick families. For CRS, separating the data sets into thin and thick families seemed to only reduce the data set which did not improve the relationships. The resulting relationship for all pavement families shown in Figure 8.8 demonstrates that if AC mix FI is increased, higher CRS values result which translates directly into longer AC pavement life. The relationships for thin and thick-pavement families are presented in Figures 8.9 and 8.10, respectively.

For IDOT, the CRS value is used as a trigger to determine eligibility of pavement sections for rehabilitation. The Figures 8.3-8.5 show that a decrease in AC mixture FI results in reduced CRS values over time. The slope of the FI/CRS relationship ranged from approximately 0.2 to 0.4. Assuming an average slope 0.3 and with the ability to limit AC mixes to a minimum FI of 8.0 (as proposed by IDOT) would increase the overall average FI for the mixes in this study approximately 3 points (FI average of 6 to average of 9 in order to meet minimum requirement). This 3-point move in FI would then translate to a 0.9 increase in the CRS value in the first few years of pavement life. Due to the limited time length of this study, it can only be assumed that the improved CRS value would be at least partly retained throughout the life of the pavement, thus increasing pavement life.

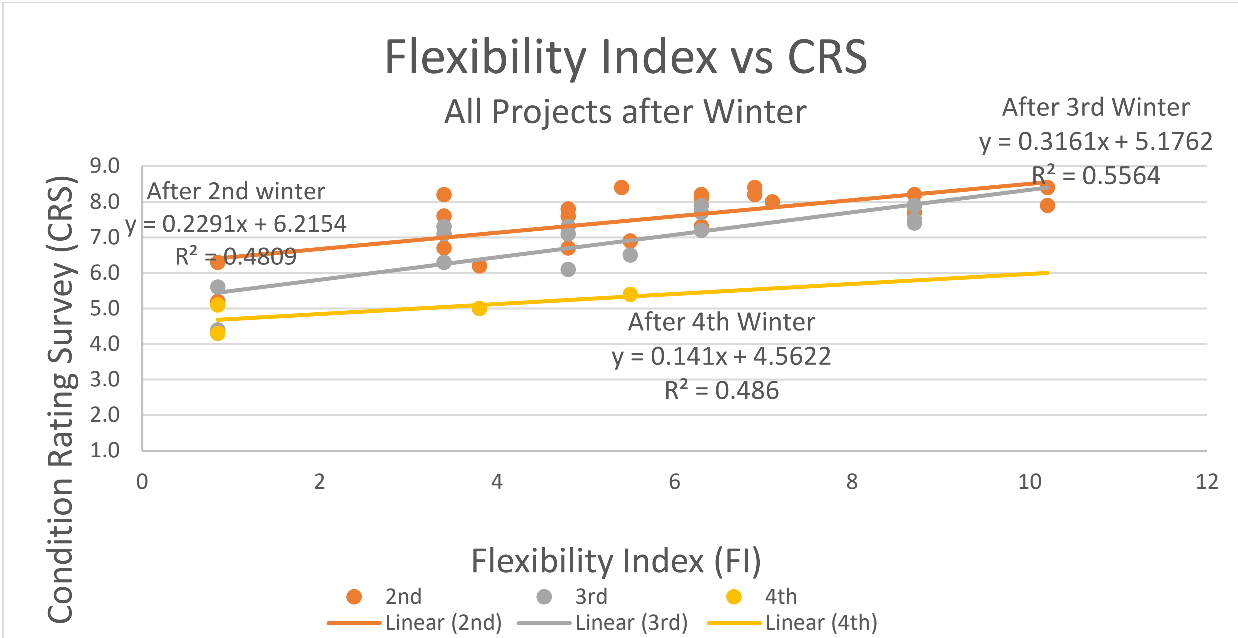


Figure 8.3. Flexibility Index (FI) relationship to Condition Rating Survey (CRS) value for all projects.

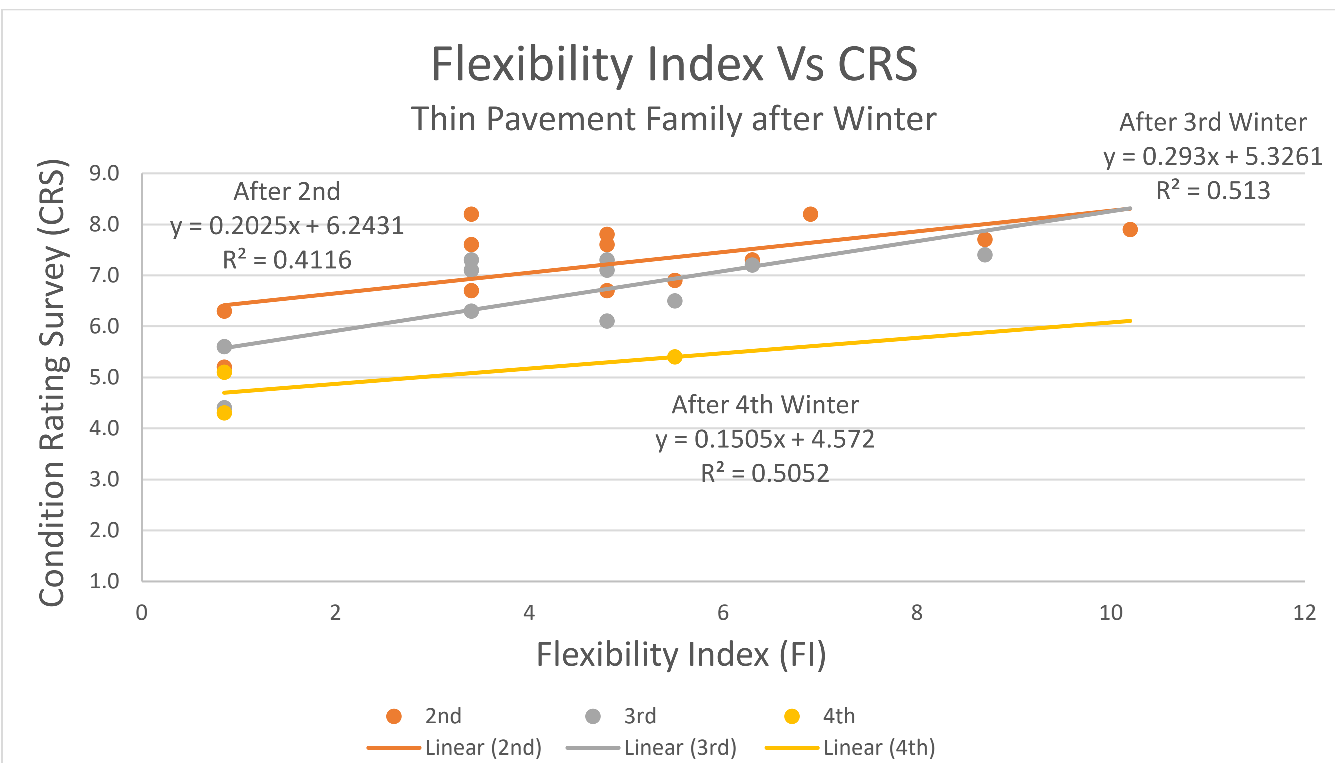


Figure 8.4. Flexibility Index (FI) relationship to Condition Rating Survey (CRS) value for thin projects.

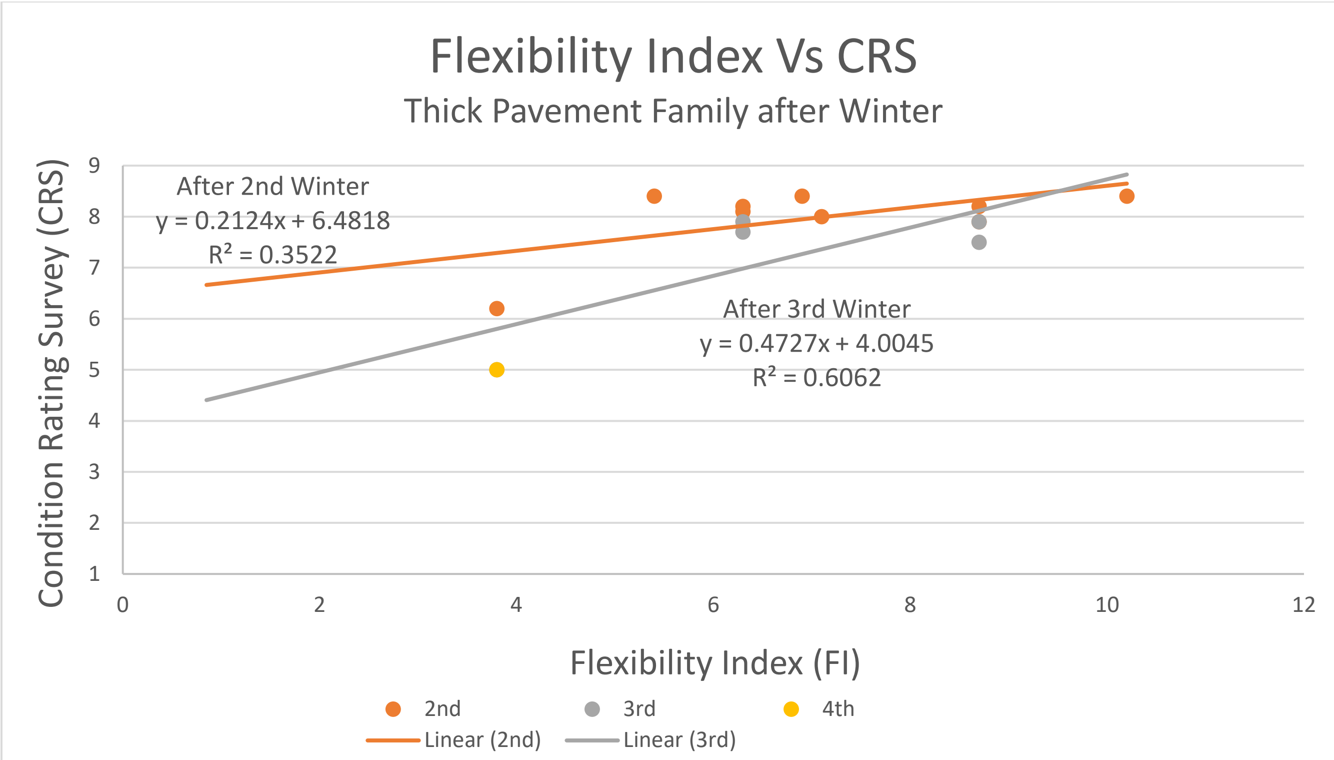


Figure 8.5. Flexibility Index (FI) relationship to Condition Rating Survey (CRS) value for thick projects

CHAPTER 9: SUMMARY, FINDINGS, AND RECOMMENDATIONS

9.1 SUMMARY

The main purpose of this study was to document pre-existing conditions, construction procedures, characterize the materials used, and monitor the resulting performance of five experimental sections plus four demonstration projects. The experiments used asphalt concrete (AC) surface mixes that contain reclaimed asphalt pavement (RAP) with and without recycled asphalt shingles (RAS) at a variety of asphalt binder replacement (ABR) levels. To counter brittle asphalt from recycled sources, various grades of PG asphalt binders that are softer than typically specified were evaluated. This report documents the performance to date of three total-recycle asphalt pavement sections and a comparison section let April 26, 2013 and five projects let June 13, 2014 by the Illinois Department of Transportation. The 2014 let projects were documented in detail from start to finish of construction to determine what influence construction may have on cracking and performance of the new AC overlay. A total of 12 mixes were evaluated which ranged in ABR from 15 to 60 percent. AC mixture properties were determined on the plant mix at production and then in-situ through pavement coring during the two to four year life span of the pavements depending on when the project was constructed.

This study found that pavement rehabilitation practices from design to construction are producing results consistent with existing policy and guidance. The plans on these projects had adequate repair quantities as a whole for patching with field staff making adjustments as needed during construction.

Milling of pavement can produce variable results; attention needs to be given to teeth condition in the mill head, as just a few new teeth in an otherwise worn head can result in deep grooves in the milled surface, which is not desirable. Although not tied to performance issues to date, excessive deep grooves can lead to yield problems and cost increases of the more costly level binder.

Pavement ride quality after construction resulted in values that were much rougher than are typically measured on in-service Interstate highways, especially in the right-wheel-path. A review of the increased roughness in the right-wheel-path compared with the left wheel path showed this phenomenon is typical of other projects in Illinois and other states. This could be due to the grade control being placed on the left side of the paver. Other possible causes are the level binder edge detail used on these projects and contractor practice of adjusting yield by slope adjustments.

Although AC characteristics are the main cause of crack development, the combined AC overlay thickness of new and pre-existing may not be overlooked. There were two main families of pavements in this study: Those that were bare-PCC to be overlaid or milled down to bare-PCC, resulting in 2.25 to 3 in of new AC overlay over the concrete pavement; and the other group of overlaid PCC pavement with much thicker combinations of new and pre-existing AC overlays, in the range of 5.75 to 8.0 in. These families were termed “thin” and “thick” respectively. The thin-pavement family reflected cracks quicker and at higher severities than the thick-pavement family.

The I-FIT Flexibility Index (FI) correlated to the rate of transverse cracking in both thin and thick-pavement families. The use of a minimum FI of 8.0 is suggested by IDOT for AC surface mixes, a value that is supported by analysis of early-age cracking in this study. The Texas overlay test was also reviewed, but the field early-cracking data and results of this test did not correlate well.

The performance of AC mixes were analyzed using the modified balanced mix design approach proposed in the earlier ICT R27-128 study. A 3-D performance diagram was used to evaluate the interactions between FI, rut depth, and stiffness obtained from the same I-FIT. The multi-criteria performance evaluation approach was proposed to avoid premature failures.

9.2 KEY FINDINGS

The followings are findings of this study:

- Transverse cracking initiation and propagation are influenced by both the AC mixture characteristics and pavement thickness family.
- Projects that left an existing AC layer of 3.5 in or more in place after milling resulted in less cracking in the new overlay, given the in-place AC is intact.
- The regression analysis of FI and transverse cracking indicates that transverse cracking can be reduced in both thin- and thick-pavement families by using AC with a minimum FI of 8.0.
- Low FI values and thin AC overlays of PCC pavement, will likely result in high amounts of reflective cracking early in the overlay life. This was the case with the Harrison Street project which was a thin overlay with the lowest FI values of the AC mixes studied (the AC mix has 56% ABR including 5% RAS). The results were the most transverse cracking at the highest severity of the projects studied.
- FI values meeting a minimum value of 8.0 are obtainable as demonstrated in the Washington Street project, which used a soft asphalt binder (PG 58-34) along with a moderate asphalt binder replacement (ABR) rate of 30% resulting in a FI just over 10.
- The values of secant modulus used in the 3-D performance diagrams ranged from 3.5 to 8.3 kN/mm. The AC mixes with highest FI has secant modulus values between 3.5 and 5.0. Stiffness may not be a controlling criterion within the balanced mix design approach, in this study, since all of the AC mixes used in this study are relatively stiff (also shown with the rut depth values all below 5.0 mm)

- Increased FI values correlated to higher Condition Rating Survey (CRS) values as the pavement aged and therefore longer pavement life.
- The Illinois Flexibility Index Test (I-FIT) and resulting flexibility index (FI) can be used on plant-sampled laboratory compacted AC mixtures to effectively characterize the potential of transverse cracking in pavements.
- The use of polymer asphalt binder (PG 70-28) in the 4.75 level binder with approximately 30% asphalt binder replacement from RAP and RAS resulted in FI values similar to the surface AC mixes in this study, which may negate the purpose of using level binder to control reflective cracking. If a level binder is used in a project not just to correct surface deficiencies and provide a platform for surface overlay construction, but also to control crack mitigation, much higher FI values are needed for the level binder AC mixture.
- The Texas overlay test did not correlate to transverse cracking development in the first four years.
- Pavement rutting is well within values that would be expected for the pavements under study.
- For the 2013 let TRA projects, pavement distress, (types, extent, and severity) is developing sooner than for the comparison project on Wolf Road. The net effect is that the pavement distress on these TRA projects is more advanced, as if the TRA pavement was two or more years older than Wolf Road. TRA projects contained 37 to 60% ABR, while Wolf Road project has 20% ABR with no RAS.
- The TRA projects on US 52 (60N07 and 60N08) are performing much better than the TRA projects constructed in 2013, which correlates to FI properties of the AC mixture and pavement families.
- Specific to TRA mixes, the use of slags and crushed concrete with high RAP values, can result in mix-control issues, as seen on US 52 contract 60N07 which needed multiple test strip trials to receive approval of the AC mix. Use of virgin aggregates would result in better control of the mix and less absorption of the asphalt binder.
- Proper tack coat application is essential to ensure bonding between layers. The placement of tack coat was adequate for all projects in this study.
- Patching plan quantity was adequate for the required patching at time of construction for the projects in this study.

9.3 RECOMMENDATIONS

The Illinois Flexibility Index Test (I-FIT) should be adopted for use as a specification requirement in AC mixture design and/or production.

The proposed Flexibility Index (FI) value of 8.0 by the Illinois Department of Transportation should be adopted. The result would be a significant reduction of cracking and crack severity during the early life of AC pavements. A performance related test would be an improvement over method specifications limiting asphalt binder replacement (ABR) and related asphalt binder grade adjustments currently in use. Desired FI value is obtainable with proper selection of asphalt binder grade, asphalt content and recycled material type and content.

In conjunction with the recommended FI controls, the use of Hamburg Wheel Track testing should be continued. Together, I-FIT and Hamburg Wheel-Tracking specification controls provide a balanced mix design with reduced risk of both rutting and cracking. The use of secant modulus, as a third performance criterion is recommended, especially that no extra testing is required. The results are readily available from I-FIT, which can be provided in the software. It would be up to the agency to specify use of the result.

Alternatives to the “mill-and-fill” approach of pavement rehabilitation such as hot or cold in-place recycling should be considered where appropriate in order to obtain the benefits of reduced cracking of the “thick” pavement family.

Alternatives to the 4.75 level binder mix used in these projects should be researched specifically to lower cost and increase the resulting FI, while maintaining the workmanship properties of this mix. A FI value for level binder of greater than 15 should be selected as a goal in order for this pavement layer to provide a crack mitigation benefit.

Pavement smoothness policy and specifications use should be reviewed with the goal of reducing pavement roughness on new overlays of major collector and arterial roadways. The 16 ft straightedge should be limited to low-volume/low-speed roadways.

Continue pavement distress data collection of study pavements. Additional data collection would provide information on the ability of I-FIT to predict cracking performance beyond the first few years of pavement life.

The need for partial-width use of level binder should be re-evaluated. If partial-width use of level binder is to continue, a tapered-edge detail by hand luting should be considered.

Training and technology transfer opportunities should be used to insure milling operations result in acceptable surface textures with respect to pavement grooving from differential tooth wear.

9.3.1 Recommendations from 2015 Interim Report

Coring existing AC overlay to determine depth of cracks and existing AC layers, allows adjusting mill depth and would assist in reducing thin AC layers that degrade under traffic.

Adopting cold-milling specifications that limit the variability of the milled surface should be considered. Such specifications are used by other states and are available.

Ensure tack coat application uniformity. The amount of “zebra striping” observed on these projects would be considered the maximum limit that would be allowed.

Patch any cracks/joints wider than 2 in and with a length of 3 or more ft.

Re-evaluate the need for partial-width use of level binder. If partial-width use of level binder is to continue, consider adding a tapered-edge detail by hand luting.

Use grade reference devices of adequate length and consider using them on both sides of the paver. At a minimum, discourage “chasing yield” and implement cross-slope controls.

9.3.2 Recommendations from 2016 Interim Report

Building up AC over underlying concrete pavement over time should be evaluated. Allowing sound material to remain on lower volume roadways may be one option. However, strong assurances would be needed through testing/evaluation so that any material left in place would not result in future rutting/stripping issues. As an alternative, in-place recycling may assist in providing additional thickness over concrete pavement to assist in reducing reflective cracking.

Thicker level binder lifts along with improved FI values for AC over bare-concrete pavements should be evaluated for cost and long-term performance.

Evaluation of a more appropriate FI value for level binder should be considered. The use of a higher FI mix below the surface would make the overlay more crack resistant. A review of the benefit/costs of FI values up to double what may be selected for the surface should be examined. A more economical level binder with improved FI properties should be the goal.

While the use of thin overlays using high recycle content materials may seem desirable to reduce cost, more sustainable pavements may be obtained through higher FI AC mixes that are designed with a slightly thicker overlay. The economic trade-offs among thickness, FI, polymer use, and overall performance need closer examination to optimize life-cycle cost and performance.

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