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Summary Findings of Re-Engineered Continuously Reinforced Concrete Pavement: Volume 1

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16. Abstract

This research project conducted laboratory testing on the design and impact of internal curing on concrete paving mixtures with supplementary cementitious materials and evaluated field test sections for the performance of crack properties and CRCP structure under environmental and FWD loading. Three experimental CRCP sections on Illinois Route 390 near Itasca, IL and two continuously reinforced concrete beams at UIUC ATREL test facilities were constructed and monitored. Erodibility testing was performed on foundation materials to determine the likelihood of certain combinations of materials as suitable base/subbase layers. A new post-tensioning system for CRCP was also evaluated for increased performance and cost-effectiveness. This report volume summarizes the three year research effort evaluating design, material, and construction features that have the potential for reducing the initial cost of CRCP without compromising its long-term performance.

17. Key Words CRCP, Internal Curing, Lightweight Aggregates, Macro-Fibers, Active Crack, Steel, Superabsorbent Polymers, Hydration, Shrinkage, Relative Humidity, Post Tensioning, Performance, Erosion, Foundation Layers, Construction	**18. Distribution Statement** No restrictions. This document is available through the National Technical Information Service, Springfield, VA 22161.		
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EXECUTIVE SUMMARY

Reconstruction of high volume corridors requires a pavement with an extended performance life, minimal maintenance, exceptional smoothness, and superb durability. Continuously reinforced concrete pavement (CRCP) has historically offered these characteristics and provide advantages over other pavement types in terms of prolonged smoothness, greater maintenance intervals, and the most favorable concrete surface for a future asphalt overlay. CRCP has been used for over 50 years on high volume expressways in the Chicago area with outstanding service life performance.

Initial costs associated with the steel reinforcement and placement, empirical structural designs that produce a slab thickness similar to JPCP, past experience with concrete material-related distress associated with CRCP design, undesirable cracking patterns, and erodibility of the base/subbase layer are the main factors that need to be re-engineered for high performing and cost-effective CRCP. The objectives of this research study was to provide the Illinois Tollway with tested solutions that give a balanced CRCP system having the same or lower initial cost and life-cycle cost assessment relative to other pavement options. These objectives could be achieved through innovations in the structural design, concrete material constituents and proportions, construction processes (curing management, active crack control, terminal joint design, etc.), and support layer selection without sacrificing the desired early and long-term performance.

The project's research approach was to conduct laboratory testing on the design and impact of internal curing on concrete paving mixtures with supplementary cementitious materials (SCM) and construct field test sections to observe the performance of the crack properties and CRCP structure under environmental and FWD loading. Three experimental CRCP sections on Illinois Route 390 near Itasca, IL and two continuously reinforced concrete beams at UIUC ATREL test facilities were constructed and monitored. Erodibility testing was performed on foundation materials to determine the likelihood of certain combinations of materials as suitable base/subbase layers.

The field sections found that internal curing on average decreased CRCP crack width by 25% compared to non-internal curing concrete. Internal curing also reduces the number of undesirable cracks but did not eliminate them. Significant reduction in slab thickness cannot be expected by addition of internal curing alone. Active crack control in addition to internal curing almost eliminated undesirable cracks appearing on the CRCP. In addition, active cracking created straighter and tighter cracks at regular spacing. When active cracking was done throughout the length, the notches activated at early age. Inclusion of macro-fibers provide tighter crack widths especially at lower steel content. Steel content was the most significant factor in controlling crack spacing and crack width and therefore selection of design content is extremely important for CRCP performance. The application of black bar did not affect the short-term performance in terms of cracking properties nor is it likely to produce long-term corrosion issues given the depth of steel and width of cracks. Performance-based specifications are needed for proportioning paving mixtures for internal curing, lower cement content, and increased SCM content.

Slab-base friction provides several inches of equivalent CRCP slab especially when utilizing a cement treated base (CTB) layer. The research did not find significant difference between different interlayer

types on top of the CTB. The new terminal joint design adopted in the project was simpler to construct and active cracking at the ends adds the potential for reducing the end movements expected at the end of CRCP. The transverse post-tensioning study of CRCP demonstrated acceptable transverse strain distribution in steel and concrete stress from the preliminary results. Transverse curling was also reduced as a result of transverse post-tensioning of the CRCP.

The research study findings are described in detail in four main reports (volumes 2 to 5) with volume 1 being a summary of the significant research findings for the entire research effort. Volumes 2 to 5 provide details and findings about the field test section performance, FWD backcalculation results, concrete mixture constituents and proportioning, and construction processes and foundation layer erodibility performance.

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CHAPTER 1: INTRODUCTION

1.1 PROJECT BACKGROUND

Reconstruction of high volume corridors requires a pavement with an extended performance life, minimal maintenance, exceptional smoothness, and superb durability. Continuously reinforced concrete pavement (CRCP) offers these characteristics and provides advantages over jointed plain concrete pavement (JPCP) and asphalt concrete with prolonged smoothness, greater maintenance intervals, and the most favorable concrete surface for a future overlay. CRCP has historically been used on high volume roadways in the Chicago area with outstanding performance (Tayabji et al. 1999; Gharaibeh et al. 1999), as well as in other states such as Texas, Virginia, Oregon, California, and recently Indiana. The original CRCP designs on IDOT's Stevenson, Edens, and Dan Ryan Expressways accomodated more traffic than they were designed for and provided a pavement structure for future overlays (Gharaibeh et al. 1999; Gharaibeh and Darter 2003). Illinois DOT's reconstruction of the Kennedy expressway (1992 to 1995) utilized CRCP as well as the reconstruction of the Dan Ryan Expressway from 2006-2008. In recent years, the Illinois Tollway has applied CRCP as part of reconstruction solutions on I-294. CRCP was constructed on a 5-mile section of I-294 from I-80 to the Indiana border in 2005 (slab thickness = 12 inch and steel content = 0.80%) and a 11.3 mile CRCP project on I-294 from 159th Street to 95th Street in 2008-09 (slab thickness = 12 inch and steel content = 0.80%).

The main limitation of reconstructing with CRCP are the initial costs associated with the steel reinforcement and placement, structural designs that produce a slab thickness similar to JPCP, past experience with concrete material-related distress associated with CRCP design, and erodibility of the base/subbase layer. Besides providing a competitive life cycle cost (Roesler et al 2016), CRCP has also shown to provide a lower life cycle assessment (LCA) with respect to sustainability (Ferrebee et al 2018). CRCP performance has been strongly linked to its transverse crack spacing and width, which are affected by the concrete material selection, climate, base friction, longitudinal joint types, and concrete-steel interaction and configuration. Premature failures of CRCP have occurred because of concrete material-related distress and subbase erosion (Gharaibeh et al. 1999; Zollinger and Barenberg 1990) and other CRCP failure modes include punchout from repeated loading and variation from best design and construction practices (Roesler et al 2016). Furthermore, terminal or transition joints such as wide flange and lug systems are expensive to build and maintain and must be addressed in a roadway with a significant amount of bridge structures.

Recent research has focused primarily on the structural design of CRCP (ARA 2003; AASHTO 2008; Roesler and Hiller 2013), subbase erodibility analysis and testing (Jung et al 2010), and active crack control (McCullough et al 1999; Kohler and Roesler 2004; Ren et al 2014). Internal curing with lightweight aggregates (Weiss and Lura. 2012; Bentz and Weiss 2011) has been used in concept for several CRCP test section in Texas (Ledbetter et al 1965; Friggle and Reeves 2008; Rao and Darter 2013), but no detailed analysis and testing has been conducted for its optimal use in CRCP. The current version of AASHTO Pavement ME (2011) has an improved CRCP punchout model but does not directly consider extended lanes, subbase performance properties and erodibility tests, transverse post-tensioning, internal curing, higher percentages of SCM, CRCP terminal joints (Jung et al. 2007), and optimal slab-base frictional properties. Automatic placement of steel called tube feeding has also been done in the past (Roesler et al. 2016), but not without links to premature CRCP failures in some projects. Likewise, concrete curing is critical for the development of early and long-term properties and transverse cracks in the CRCP, but modern curing products and monitoring practices still need significant improvement to become more active instead of passive.

Recent designs in Belgium and for the North Texas Tollway have demonstrated that a more economical CRCP can be built without sacrificing performance. Re-engineering, innovating, and building a more cost-effective CRCP will require use of recycled and by-product materials in various pavement foundation layers, new construction processes to place steel and activate cracks at the desired intervals, more erosion-resistant support layers containing recycled materials from previous roadways, structural design of the CRCP slab thickness with the AASHTO Pavement ME and appropriate inputs, prediction of crack spacing and width with greater use of supplementary cementitious materials (SCM) and internal curing, and alternative terminal joints.

1.2 RESEARCH OBJECTIVES

The research project objectives were to provide the Illinois Tollway with tested solutions that give a balanced CRCP system having the same or lower initial cost and life-cycle cost assessment relative to JPCP and asphalt concrete. These objectives can be achieved through innovations in the structural design, concrete mixture constituents and proportions, construction process improvements (curing management, active cracking, etc.), and support layer selection without sacrificing the desired early and long-term performance. Any innovative solutions to make CRCP more economically viable must also maintain the same service life of the CRCP under repeated loading by also maintaining the physical and chemical durability of the concrete and steel. The traditional CRCP slab thickness for heavier traffic routes (12 to 14 inches) will need to be decreased by multiple means such as smaller designed crack widths, internal curing (IC), transverse post-tensioning, and a more erosion-resistant support layer that utilizes the existing construction materials. Improvement in CRCP construction efficiency will be required to further increase LCCA and LCA benefits such as a more efficient steel placement, improved curing management, active crack control, and a better terminal joint design. This research project has undertaken extensive laboratory and field testing of re-engineered CRCP test sections at the UIUC ATREL facility and the Illinois Tollway to assist in the design of viable CRCP options that lead to the LCCA and LCA goals.

CHAPTER 2: LABORATORY EXPERIMENTS AND FIELD TEST SECTIONS

The research project involved laboratory and field investigations of various new ideas to provide a balanced CRCP system given an existing roadway corridor that needs reconstruction. Companion volumes of the report provide the details about all the tests. This section provides a brief overview of the laboratory and field testings involved in the multi-university research project.

2.1 LABORATORY TESTING

2.1.1 Concrete Mixture Proportions, Constituents, and Durability

The goal of this laboratory research was to examine the potential for using alternative materials to enhance the binder that is used in the CRCP and to reduce the potential for curling and warping related to differential moisture movement as well as thermal effects. The initial research measured moisture distribution using small scale tests and material scale tests for the purpose of quantifying stress in sections of conventional and internally cured concrete. Additionally, chloride ingress and associated freeze-thaw was also examined for potential cementitious binders.

The first portion of this project supported the implementation of lightweight aggregate for internal curing in field paving trials. As expected, the field trials utilized a large volume of lightweight aggregate in pavements as compared with bridge decks which required a relatively large laydown area at the plant and a large area for lightweight aggregate prewetting. Research was performed to attempt to reduce the volume of lightweight aggregate used in each cubic yard of concrete or to utilize superabsorbent polymers as an alternative internal curing agent. Additionally, the impact of internal curing on the development of moisture gradients and curling was investigated through neutron radiography as well as the curing quality at near the surface of the concrete. The durability of IC mixtures under freezing and thawing and chloride ingress was also evaluated along with higher supplementary cementitious material (SCM) replacement of cement. A full description of the laboratory testing plan for internal curing mixtures, durability, and SCMs can be found in volume 4 of this research (*Examining Cost Saving Measures in Material Selection for Continuously Reinforced Concrete Pavement*) by Montanari et al. (2021)

2.1.2 Concrete-Base Interface Properties and Erodibility Performance

Both labortory and field work related to the concrete-base interface properties and erobility was carried out. The laboratory work consisted of erosion and bond testing of typical base material used by the Tollway in roadway construction. A detailed explanation of all work completed on this topic is documented in Volume 5 by Speakmon et al. (2021). Lab samples are cored from prepared slices that consisted of an interface between PCC and a base material. A circular sample was cut into two semi-circles combined of PCC/AC portions consisting of a pre-selected angle as shown in Figure 1. The fracture can be oriented along different orientations: pure tensile failure, pure shear failure, and three mixed tensile/shear failure points. The characteristics of the specimen can also be controlled by choosing different geometric properties: radius (R), length of the notch (α), angle of the notch (δ), and the distance of support (1) and (2) from the center (S_1) and (S_2) as seen in Figure 1.

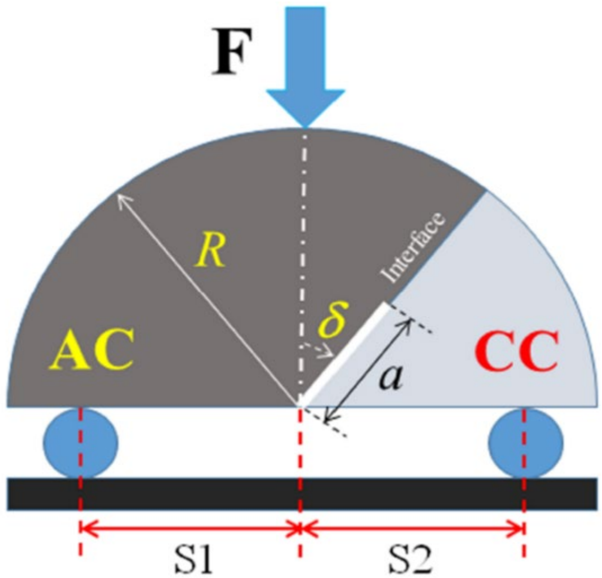


Figure 1. Semi-circular specimen.

A laboratory testing by this approach was useful to investigate the effects for four prepared specimens consisting of four interface treatments between an asphalt base and concrete layers. The results of the laboratory testing is illustrated in Figure 2 where the interfacial treatments consisted of:

- No bond breaker (BB)
- 1 coat of resin curing compound (RES1x)
- 2 coats of resin curing compound (RES2x)
- 1 coat of an emulsion surface treatment (SS1)
- 2 coats of a wax-based curing compound (WAX2x)

Figure 2 results also indicate the amount or the percent reduction in bond strength because of the different interfacial treatments. Erosion testing was carried out for the following base materials:

- Bank run
- PGE
- CA-6
- RAP Cap
- Inverted prime Interlayer

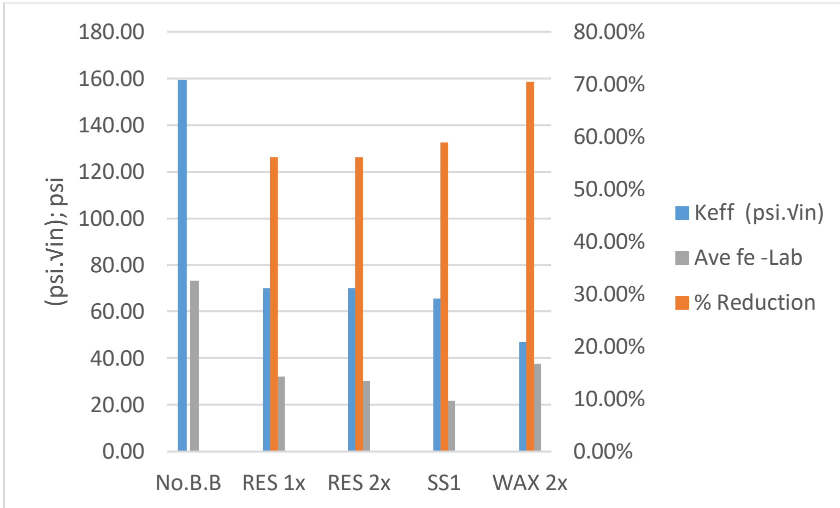


Figure 2. Test results from interfacial fracture laboratory testing of four interface treatments.

The bank run and the PGE (porous granular embankment) had fairly broad gradation bands with limited fines while the CA-6 met Illinois DOT specifications for an unbound base layer. The RAP Cap material was made up from recycled asphalt pavement. The inverted prime consisted of an emulsion spray on a fine graded sandy material. Erosion testing was carried out in the laboratory using the Hamburg Wheel Loading Device (HWLD) shown in Figure 3. A test specimen was made from each material. The load device creates a wear and pumping action which erodes the test specimen under each load application.

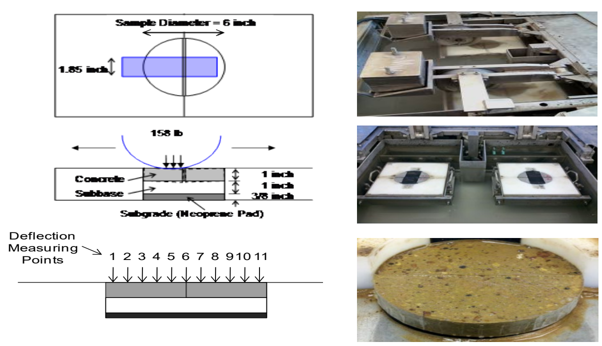


Figure 3. HWLD configuration for base layer testing.

2.2 CONTINUOUSLY REINFORCED CONCRETE BEAM SECTIONS AT ATREL

Two continuously reinforced concrete beams (CRCB) of length approximately 500 ft. were constructed on top of asphalt base layer in the ATLAS yard at ATREL. The main objectives of these CRCB sections were to study the response and performance of design feature as well as concrete material changes with respect to crack spacing and crack width development and measured strains in the concrete. Results of the field setup and observations are fully presented in Volume 2 of the report (Dahal et al. 2021).

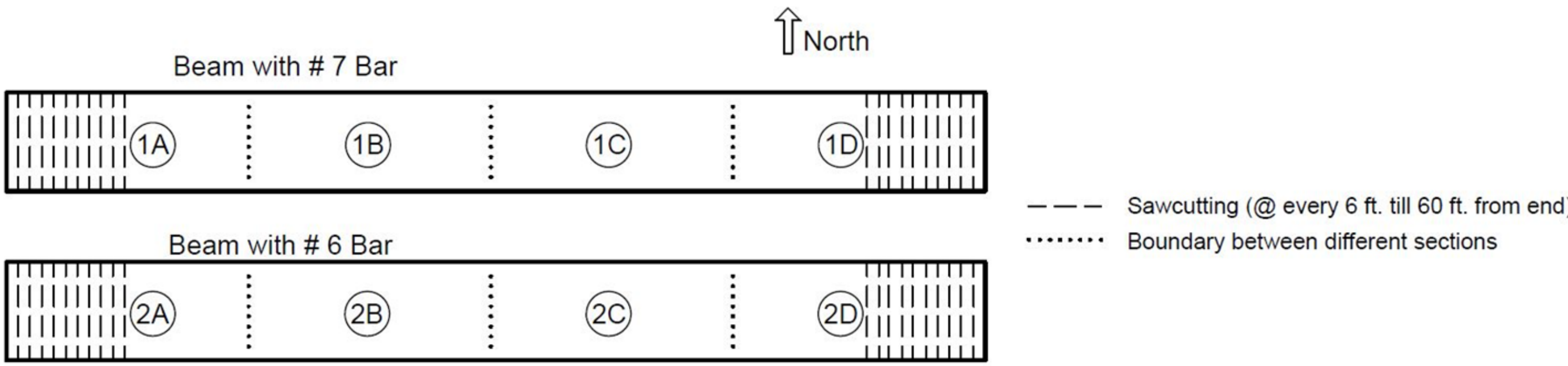
Each CRCB was divided in four sections making the sections approximately 124 ft. long. CRCB were 15 inch wide and 10.5 inch thick. Steel percentage of 0.56% (two #6 bars) and 0.76% (two #7 bars) were used with a cover depth of 3.5 inch at the top surface and edge of the beam. Two sections on each of the CRCB consisted of internally-cured section using fine lightweight aggregate (FLWA) while other two had 100% conventional (virgin) aggregates as shown in Figures 4 and 5. The CRCB with lower reinforcement also contained macro-fiber (6 lb/cy) at both ends. Ends of the CRCB were restrained to prevent excessive movements and mimic CRCP terminal joints. Both terminal ends of the CRCB were sawcut with about 2-inch notch after about 6 hours of casting to initiate active crack control. Ten notches were made at 6 ft. intervals up to the length of 60 ft. from the terminal end.

The CRCBs were instrumented using strain gauge, thermocouple, and relative humidity gauge to monitor the response over time and also to be a trial run for the sensor installation and data collection prior to application in Tollway field sections. Sensors were embedded inside the CRCB during concrete casting. A list of sensors installed in the CRCB and CRCP field section are presented in Table 1.

Table 1. List of Sensors Used in the Beams

Instrument	Manufacturer / Model	Number in each section
Strain Gauge	Micro-Measurements (EPG-5-350)	2
Strain Gauge	Texas Measurement –Tokyo Sokki (PML-60)	2
Thermocouple	Omega (Type K)	3
RH Gauge	Sensirion (SHT75)	3

Note: Details of each sensor is provided in Volume 2 (Dahal et al. 2021).



Section ID	Mixture	Reinforcement	Sawcut
Section 1A	FLWA	0.76 %	Yes
Section 1B	FLWA	0.76 %	No
Section 1C	Virgin	0.76 %	No
Section 1D	Virgin	0.76 %	Yes
Section 2A	FLWA + Fiber	0.56 %	Yes
Section 2B	FLWA	0.56 %	No
Section 2C	Virgin	0.56 %	No
Section 2D	Virgin + Fiber	0.56 %	Yes

Figure 4. Top view of beams with section name (top); nomenclature of different sections with key mixture, reinforcement, and sawcut details (bottom).

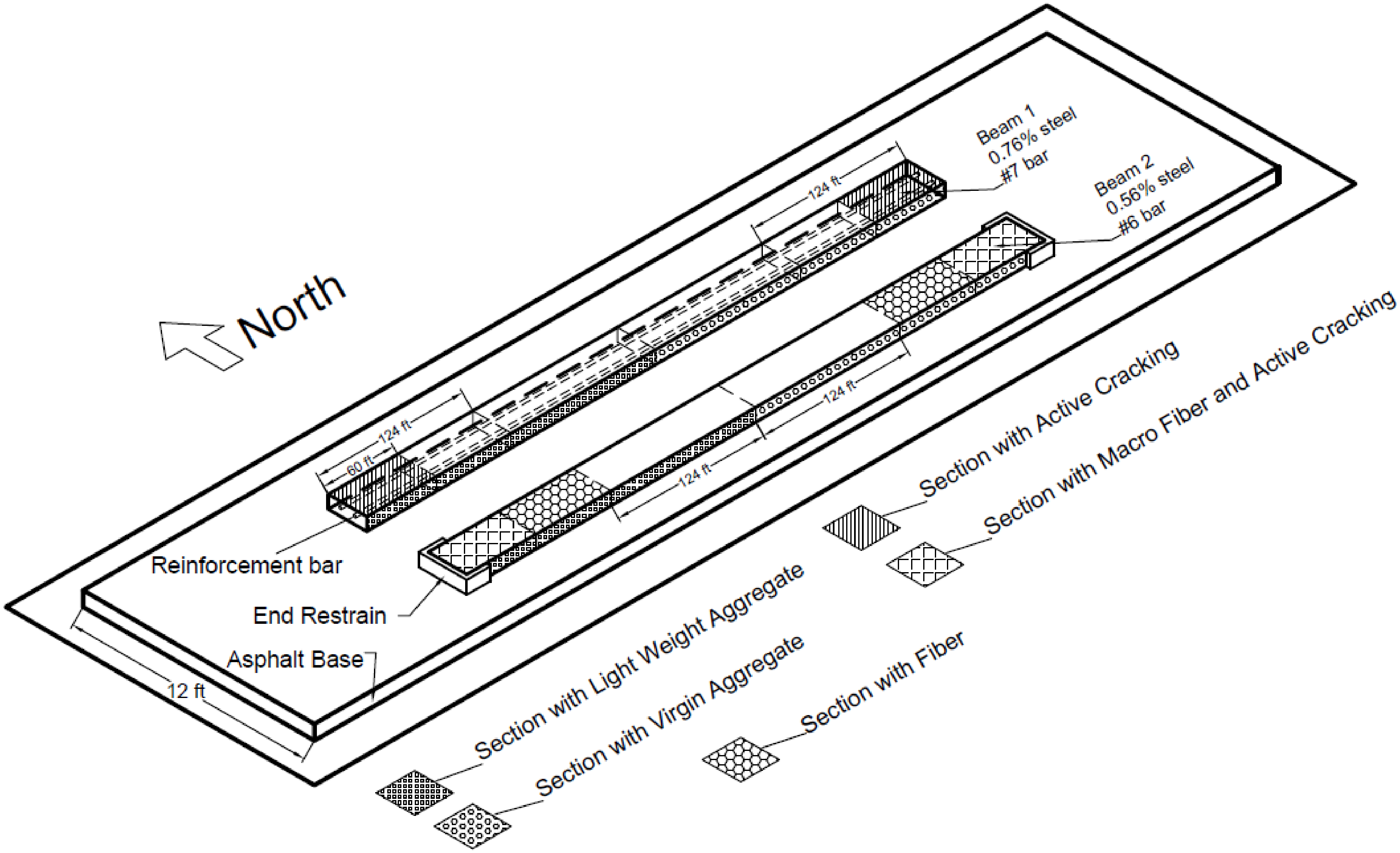


Figure 5. Isometric view of two continuously reinforced concrete beams with different design and material features.

2.3 TOLLWAY TEST SECTIONS

Three CRCP experimental test sections were constructed on the Illinois Tollway in Itasca, IL on Illinois Route 390 (also known as the Elgin-O'Hare Tollway). Two test sections were constructed in summer of 2016 while the third test section was constructed in May 2017. Each CRCP test location was further subdivided into more sub-sections through variation in base type, steel content, application of internal curing, inclusion of macro-fibers, and active crack control. For this report, the three experimental test locations are referred to as Section 1, Section 2, and Section 3. Sub-sections within each test location has a letter following the section number. For instance, Section 1D would represent sub-section D within the test section location 1.Section 1 was in the west bound lanes while Sections 2 and 3 were on the east bound lanes. Each test section consists of three lanes with the two inside lanes (closest to the median) constructed simultaneously with a slip-form paver. The widths of each lane from inside to outside are 14 ft., 12 ft., and 13 ft. respectively. Brief description of each section is presented in this chapter while details of design and material are presented in Volume 2 by Dahal et al. (2021).

2.3.1 Test Section 1

Section 1 was constructed on August 6, 2016 and was divided into five distinct sub-sections named 1A through 1E. Figure 6 presents the plan view of Section 1 with instrumentation locations with more specific details in volume 2 by Dahal et al. (2021). Table 2 summarizes the design features present in this section. This was the longest Tollway test section with 1970 ft. length. During paving of section 1, equipment malfunctioning stopped paving near section 1A resulting in an unplanned construction joint. The remaining length of section 1 was paved on August 8, 2016.

Test section 1 was nominally 10.5 inch thick and contained pre-wetted fine lightweight aggregate to provide internal curing. Internally-cured concrete pavements required similar construction processes and equipment as non-internally cured concrete pavements except the moisture content of the FLWA had to be more closely managed to maintain consistently batched mixes. Fine lightweight aggregates were incorporated by replacing about 37% of fine aggregate (pre-wetted) by volume given an aggregate absorption capacity of 10.5%. Test section 1 had variation in steel content as well as base types. Cross section with the two base types present in this test section are illustrated in Figures 7 and 8. Section 1A and 1E, which are at the two ends of the test section, consist of saw-induced transverse "joints" that are 2 ft long and 2 inch deep as shown in Figure 9. The saw-cut induced transverse cracks repeat every 4 ft. for the 50 ft. after the terminal joint at the end of the CRCP test section.

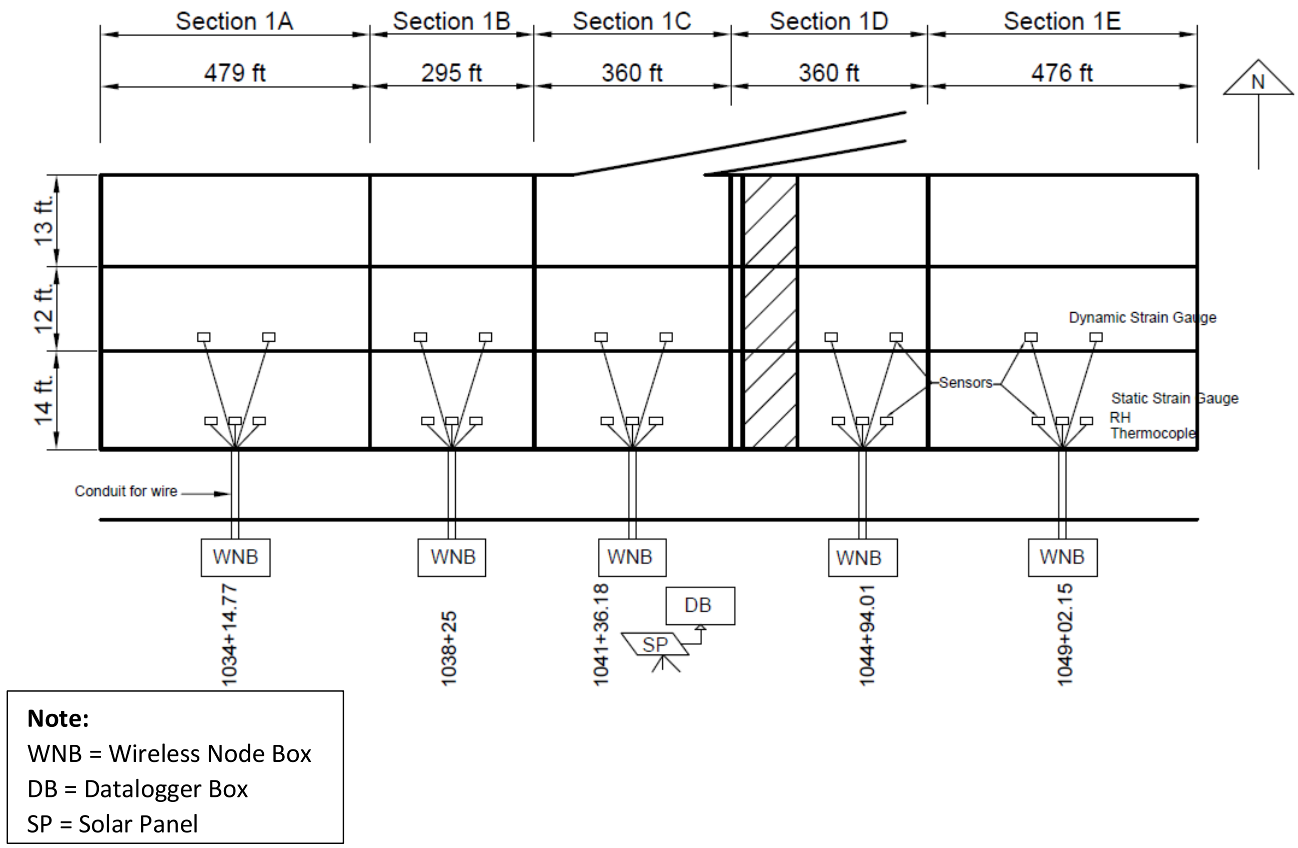


Figure 6. Plan view of section 1 (westbound) with station location of different wireless node boxes (WNB), datalogger box (DB), and sensor types.

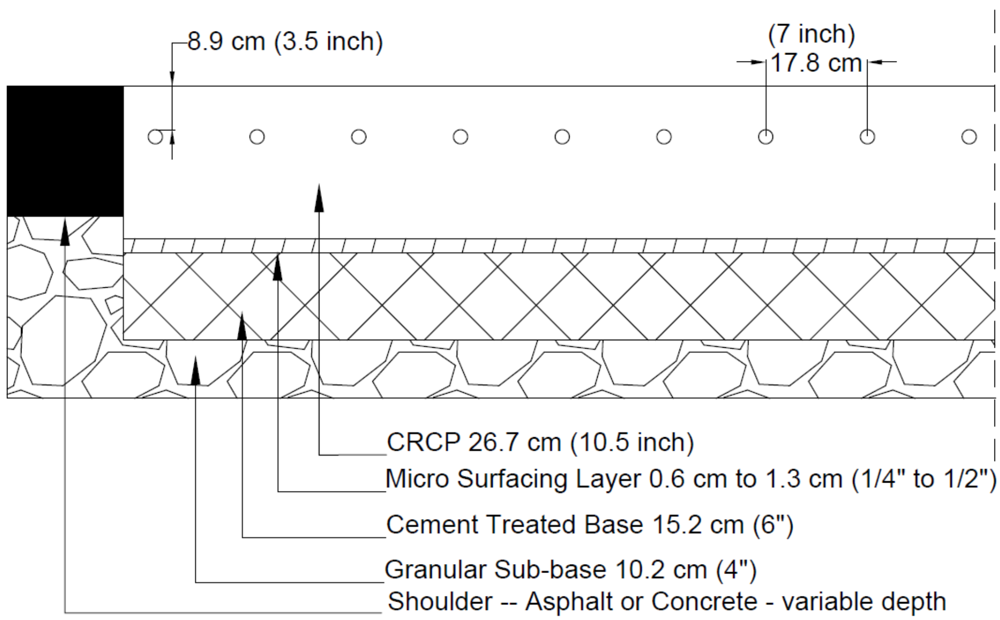


Figure 7. Cross section of sections 1A and 1B with microsurfacing layer over 6 inch cement-treated base.

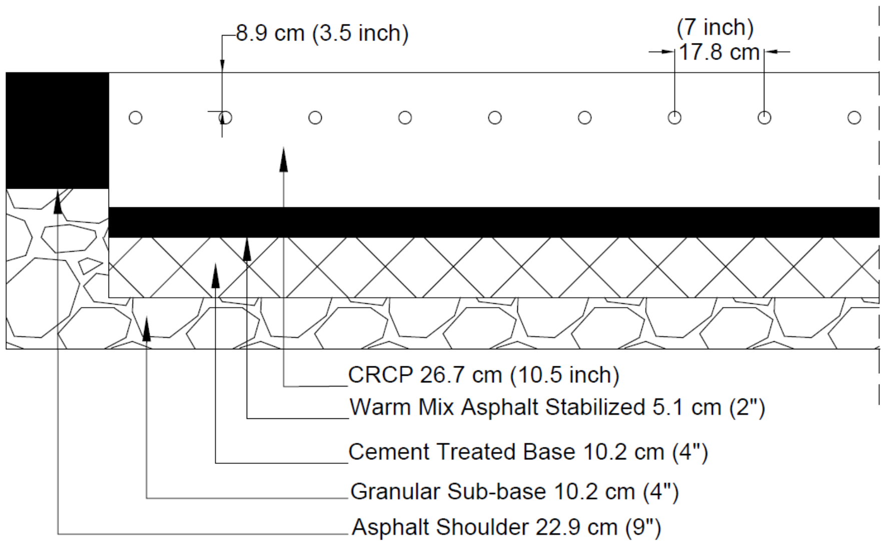


Figure 8. Cross section of sections 1C, 1D, and 1E with 2-inch warm mix asphalt over 4-inch cement treated base. The same base type is also used for sections 2A and 2B.

Figure 9. Saw-induced transverse cracks that are 2 feet long and 2 inch deep.

2.3.2 Test Section 2

Section 2 was constructed on July 15, 2016 and was 10.5 inch thick without fine lightweight aggregate for internal curing. The test section was 1044 ft. long and was divided into two subsections named 2A and 2B with the only variation being steel content (0.58% vs. 0.80%). Figure 10 presents the plan view of Section 2 with instrument locations. Table 2 summarizes the features present in section 2. The base layer of this section is presented in Figure 8.

2.3.3 Test Section 3

Section 3 was constructed on April 27, 2017. The section was 976 ft. in length with subsections 3A and 3B. This CRCP test section was 9 inch thick and contained fine lightweight aggregate providing internal curing. The subsections had two steel content (0.80% and 0.58%) with the lower steel content section containing synthetic macro-fiber (6 lb/cy). The base layers of this section, presented in Figure 11 were different than sections 1 and 2. Active crack control by partial width transverse notch induction was implemented throughout the section 3. Partial-width transverse notches 2 ft long from the edge of the pavement and 2 inch deep were induced every 4 ft throughout the length of the section. Figure 12 presents the plan view of Section 3 with instrument locations. Table 2 summarizes the material and design features present in section 3.

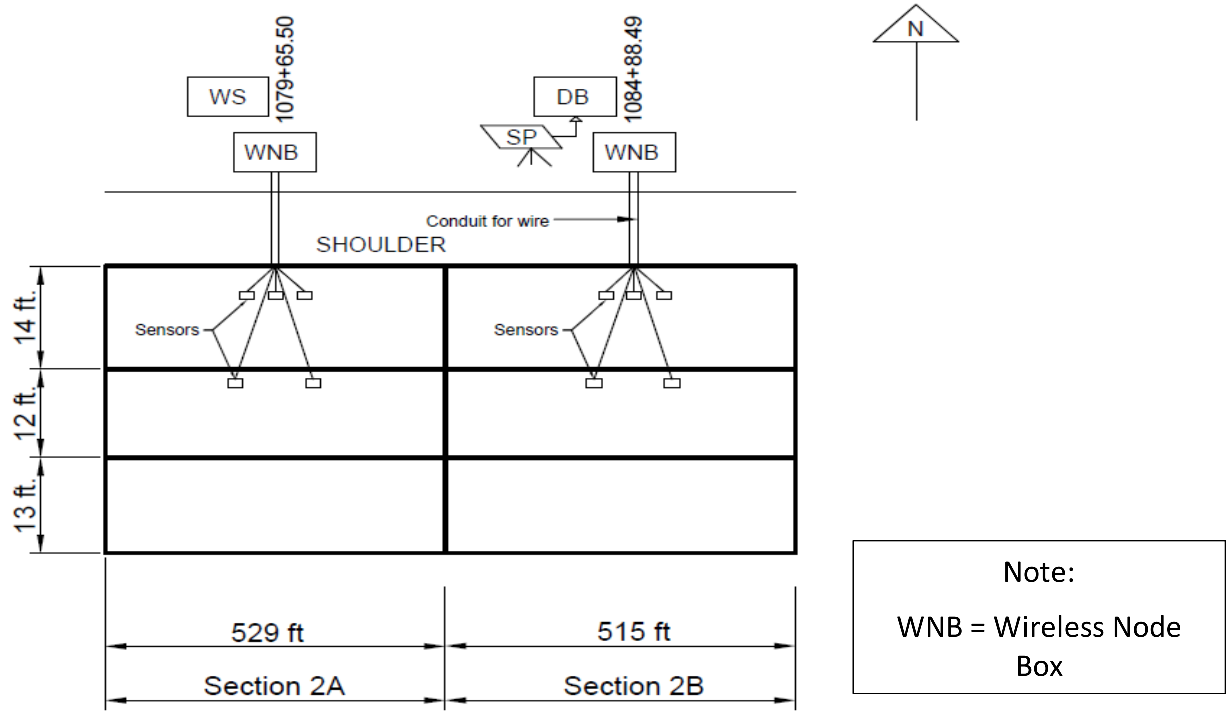
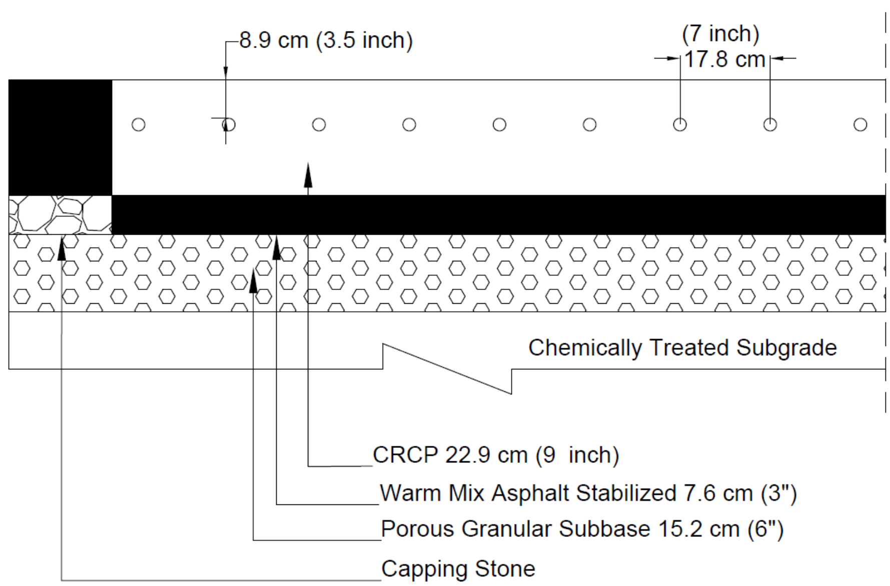


Figure 10. Plan view of section 2 (eastbound) with station location of different wireless node boxes, datalogger box, and sensor types.



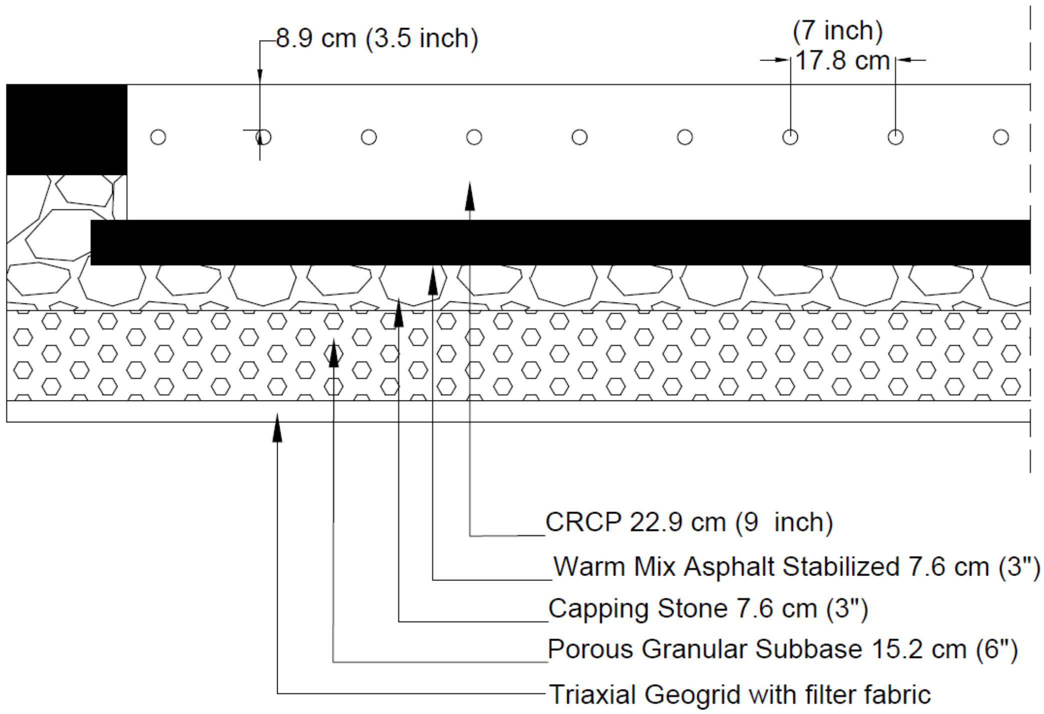


Figure 11. Cross section of sections 3A and 3B with 3-inch WMA base and various sub-base.

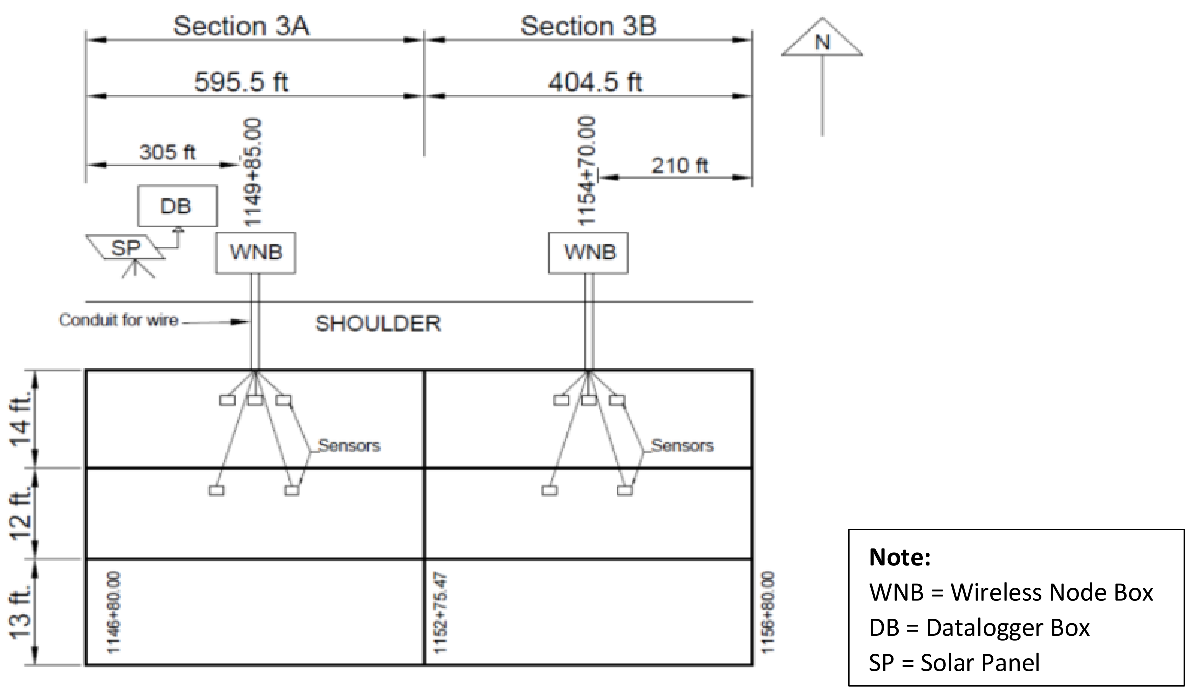


Figure 12. Plan view of section 3 (eastbound) with station location of different wireless node boxes, datalogger box, and sensor types.

Table 2. Summary of Tollway CRCP Details and Features for Sections 1, 2, and 3 on Illinois Route 390

Section (Cure / Thickness)	Sub-Section	Length, ft. (m)	Steel Content	Base/Subbase			Active Cracking – sawcut detail	
				a	b	c		
1 (IC / 10.5inch)	1A	479 (146)	0.80%	1/4" to 1/2" (0.6 cm to 1.3 cm) Microsurfacing	6" (15.2 cm) CTB	4" (10.2 cm) Granular Subbase	2ft. (0.6 m) long; 2" (5.1 cm) deep	Every 4 ft. (1.2 m) for 50 ft. (15.2 m) from end
	1B	295 (89.9)	0.58%	1/4" to 1/2" (0.6 cm to 1.3 cm) Microsurfacing	6" (15.2 cm) CTB	4" (10.2 cm) Granular Subbase	N/A	
	1C	360 (109.7)	0.58%	2" (5.1 cm) WMA Stabilized	4" (10.2 cm) CTB	4" (10.2 cm) Granular Subbase	N/A	
	1D	360 (109.7)	0.80% + Fiber	2" (5.1 cm) WMA Stabilized	4" (10.2 cm) CTB	4" (10.2 cm) Granular Subbase	N/A	
	1E	476 (145.1)	0.80%	2" (5.1 cm) WMA Stabilized	4" (10.2 cm) CTB	4" (10.2 cm) Granular Subbase	2ft. (0.6 m) long; 2" (5.1 cm) deep	Every 4 ft. (1.2 m) for 50 ft. (15.2 m) from end
2 (NIC / 10.5inch)	2A	529 (161.2)	0.80%	2" (5.1 cm) WMA Stabilized	4" (10.2 cm) CTB	4" (10.2 cm) Granular Subbase	N/A	
	2B	515 (157.0)	0.58%	2" (5.1 cm) WMA Stabilized	4" (10.2 cm) CTB	4" (10.2 cm) Granular Subbase	N/A	
3 (IC / 9inch)	3A	595 (181.4)	0.58% + Fiber	3" (7.6 cm) WMA Stabilized	6" (15.2 cm) Porous Granular Subbase	Treated subgrade only	2ft. (0.6 m) long; 2" (5.1 cm) deep	Every 4ft. (1.2 m) throughout the section length
	3B	405 (123.4)	0.80%	3" (7.6 cm) WMA Stabilized	3" (7.6 cm) Capping Stone	6" (15.2 cm) Porous Granular Embankment +Triaxial Geogrid with filter fabric	2ft. (0.6 m) long; 2" (5.1 cm) deep	Every 4ft. (1.2 m) throughout the section length

2.3.4 Testing Factorial and Test Section Nomenclature

The testing factorial that was constructed and monitored during the Tollway field project research is shown in Table 3. There are similarities within each test location, such as depth of steel or internal versus non-internal curing. A nomenclature was defined for each each section that provides general information about the section based on design and material features present as seen in Table 4. These names are referred to along with section name during the proceeding discussion in this volume.

Table 3. Testing Factorial Evaluated on the Tollway's Illinois Route 390 Field Project

Feature	Modification 1 (abbreviated name)	Modification 2 (abbreviated name)
Steel	Low (LS) – 0.58%	High (HS) – 0.80%
Internal Cure (IC)	Yes (IC)	No (NIC)
Base	Micro Surfacing + 6"CTB (MS/6CTB)	2" WMA + 4" CTB (2WMA/4CTB)
Fiber	Yes (F)	No (-)
Porous Granular	Yes (PG)	No (-)
Capping Stone	Yes (Capping)	No (-)

Table 4. Tollway Test Section Nomenclature

Section Name	Abbreviated Name
1A	HS-IC-MS/6CTB
1B	LS-IC-MS/6CTB
1C	LS-IC-2WMA/4CTB
1D	HS-IC-2WMA/4CTB-F
1E	HS-IC-2WMA/4CTB
2A	HS-NIC-2WMA/4CTB
2B	LS-NIC-2WMA/4CTB
3A	LS-IC-3WMA/6PG-F
3B	HS-IC-3WMA/3Capping/6PG

2.4 POST-TENSIONED CRCP TEST SECTIONS

A test section was established to investigate the viability of post-tensioning the transverse reinforcement in a CRC pavement project located on Ball Airport Rd, in Victoria, TX. The test section was located within a newly constructed region at the north end of the roadway. The post-tension section consisted of a 52 feet section, using 13 smooth post-tensioning transverse bars. The CRC pavement was 8-inch thick requiring 26 ft. long, smooth #6 steel bars to span continuously across the two-lane paving section. Figure 13 is an illustration of the pavement section taken in the transverse direction, prior to paving showing the standard CRC pavement steel layout at the beginning of the post-tensioning test section with the smooth bars in place shown as highlighted. A thorough explanation of the post-tensioned CRCP test section in Victoria, TX is described in Volume 5 by Speakmon et al. (2021).

The longitudinal steel was placed and tied on top of the transverse bars per normal procedure. The transverse post-tensioning bars were coated with industrial grease as a bond breaker to allow for slip between the bar and the concrete during the tensioning process. Strain data was collected with to determine the strain distribution along the bars, as well as strain data collection throughout the concrete to correlate the stress transferred to the concrete from the bars. Testing was performed to measure the forces and relaxation applied with each bar.

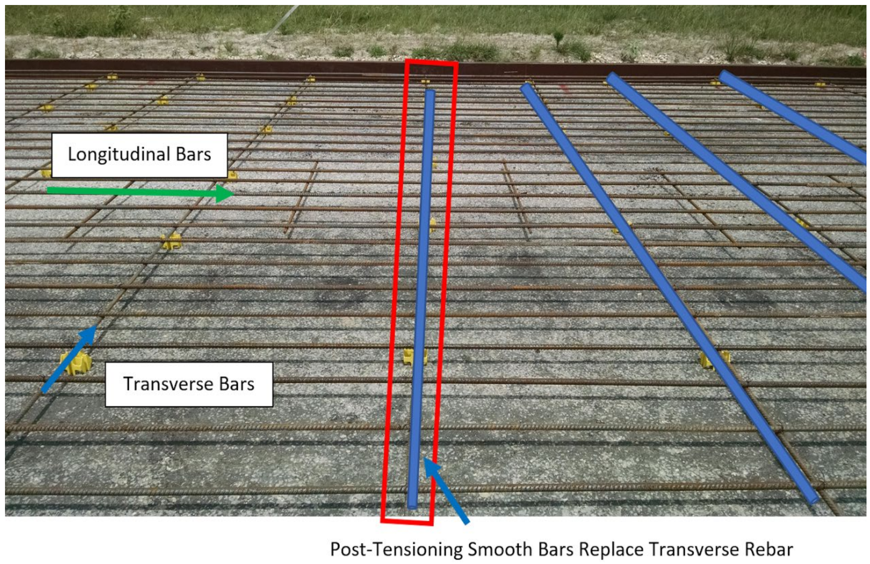


Figure 13. Project 1 (Victoria, TX) transverse rebar being replaced by smooth rigid bars.

The second PT study (Figure 14a and 14b) involved the test-slab pavement sections constructed at the Texas A&M System's Rellis Campus. This project involved creating a mock roadway segment, equivalent to the CRC section in Victoria, that consisted of the transverse rebar again replaced with smooth steel bars that were to be post-tensioned. The post-tensioning was done with the threaded bar system with the goal better modeling the stress distribution throughout the post-tensioning system. The pavement sections were originally designed to be identical to the Victoria test section, being an 8 inch CRC pavement that is two lanes wide or 26 feet. The slabs were constructed with the dimensions of 26 feet wide (simulate the transverse direction), 7 feet long (6 feet instrumented), and 11 inches thick. Both slabs were constructed on the same base layer that was placed on a standard granular sand, which best represented no bonding to the base layer and allowed for any movement or curling with minimal restrictions. Two slabs were created using standard 4,000 psi concrete with #6 smooth transverse bars. The bars consisted of two 26-foot long bars and four 13-foot bars, which combined covered all three post-tensioning designs.

Figure 14a. Test slab placement.

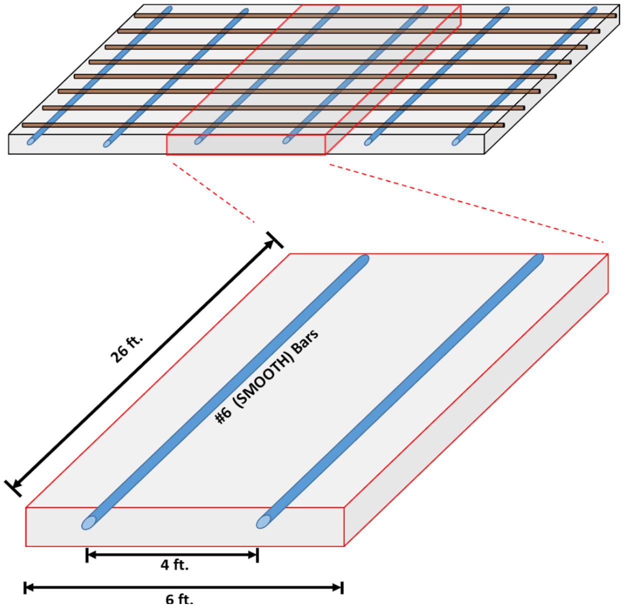


Figure 14b. Test slab simulating Victoria roadway conditions.

These test sections provided valuable data as to the viability and utility of post-tensioning of CRC pavement structures. The benefits of such a design measure is multi-faceted and provides one of the most cost effective methods to extend pavement life. It not only can lead to reduced design thicknesses but minimize the negative effects of built-in set and longitudinal joint stiffness.

CHAPTER 3: CONCRETE MATERIALS

3.1 INTERNAL CURING

One major concrete material component of this study was evaluating positive performance results from Texas when applying fine lightweight aggregates (FLWA) to CRCP. One of the first uses of FLWA in concrete pavements was in Texas in the 1960s (Ledbetter et al. 1965; Ledbetter and Buth 1970). While not specifically used for internal curing purposes, the researchers found that the incorporation of FLWA into the concrete mixture, along with coarse lightweight aggregate, the strength was enhanced and the required design thickness was lower than traditional CRCP. However, the benefits of internal curing observed were not necessarily attributed correctly.

Another comparison site was constructed in Texas in 1964 as part of State Highway 610. The CRCP sections were smaller than 500 meter in length but the normal weight and LWA concrete pavements were constructed adjacent to each other with the same base and subgrade as well as similar traffic levels. After 18 years of traffic, a crack survey was performed (Bissett 1984) and the results indicated significant performance benefits in the LWA concrete pavements sections in terms of cracking. The LWA section had much lower total number of cracks and of those a significant amount were hairline cracks. The presence of numerous hairline cracks likely reduced the pavement stress levels and prevented the development of larger cracks and associated deterioration. A follow up study at 24 years (Won et al. 1989) and 34 years (Sarkar 1999) found that the section made using LWA was still performing significantly better than the conventional concrete mixture section.

The first few Texas field test sites did not specifically characterize the effects of internal curing. In November 2006, TxDOT constructed a project (1,000 m^3) on State Highway 121 to specifically look at the effects the additional moisture on the CRCP cracking properties. The mixtures containing 178 kg/m^3 of FLWA which was sufficient amount to satisfy the estimated autogenous shrinkage amount. This site is relatively unique in that a companion section was cast alongside with conventional concrete mixture. Friggle and Reeves (2008) surveyed the crack widths in the FLWA section and found they were significantly smaller than the adjacent, conventional concrete section.

Given this experience to date, saturated FLWA can be potentially used in CRCP sections to reduce autogeneous shrinkage effects and near surface drying shrinkage through release of internal curing (IC) water as the microstructure demands. In addition, FLWAs can also be used for reducing the potential for early-age cracking, reduce the crack width magnitude for a given crack spacing, increase the degree of hydration of cement and, possibly, the degree of reaction of SCMs, with respect to non-internally cured systems.

3.1.1 Internal Curing with Fine Lightweight Aggregates

The application of fine lightweight aggregates (FLWA) to the Illinois Tollway observed that a much larger volume of FLWA was used in the pavement mixtures and the proportions were primarily based on previous experience from other FLWA applications. While the proportions are similar to that used in bridge decks, the volume of concrete pavements provide batching challenges for producers that must be managed properly. First, the FLWA volume requires a relatively large area for storage at the batch plant (See Figure 15). Second, sufficient moisture conditioning and quality control assessment of the FLWA are essential for mix delivery uniformity. Field observations by the research team (Montanari et al 2021) indicated that there was potential for inadequate moisture distribution in the FLWA stockpiles, which would make batching more difficult as the day progressed. More regular monitoring of FLWA moisture levels and making mix adjustments are essential for good quality control of this new type of paving mixture. Figure 16 shows centrifuge used for obtaining surface moisture of the FLWA (Montanari et al. 2021). It was also hypothesized that the amount of FLWA could actually be reduced while obtaining the same IC benefits thereby improving both construction process and cost benefit analysis for the concrete mixtures. A new proportioning method could divide the water into its use exact concrete demands and therefore, result in a reduction in the volume of FLWA.

Figure 15. FLWA stockpile (Montanari et al. 2021).

Figure 16. Centrifuge equipment and 3000 g bowl for spinning the FLWA to obtain surface moisture (Montanari et al. 2021).

The field trials confirmed that sufficient moisture conditioning and quality control of aggregate moisture is required and needs managing by the contractor. In the field trials, the aggregate moisture was found to vary more in this project than in typical bridge deck projects because of inconsistent aggregate prewetting partially as a result of the large volume of material being utilized. As such, it is highly recommended that the centrifuge method be required for quality control. It is recommended that the Tollway require the producer to maintain records of aggregate moisture and to properly adjust the moisture throughout the batching processes.

The Snyder and Bentz (1999) equation determines IC water demand based on the amount of chemical shrinkage of the paste and generally requires excessive amount of added water (and FLWA) that is not necessarily needed for concrete pavements. An alternative methodology for determining the FLWA replacement volume should use autogenous shrinkage and relative humidity measurements such that only a fraction of the internal curing water, provided by the FLWA, ends up filling the pores. The remaining water is accounted for by other phenomena, such as increased degree of hydration, partial desorption of the IC agent, and partial chemical shrinkage at early ages.

The proposed method is based on providing internal curing water to maintain a targeted relative humidity of the hardened cement paste by considering the paste's pore size distribution. The measurements on a mortar system show benefits in terms of increased relative humidity and reduction in autogenous shrinkage, even for the lowest replacement levels of FLWA, with higher benefits coming from higher volumes of FLWA. The pore size distribution of the matrix was measured using desorption isotherms and used to establish the volume of internal curing water needed to fill the porous medium up to a target pore radius. An expression was developed in order to predict the shrinkage reduction that can be obtain through the addition of partial volumes of internal curing water with respect to the commonly used approach. Details of this new proportioning method are presented in volume 4 by Montanari et al (2021).

3.1.2 Internal Curing with Superabrosbent Polymer

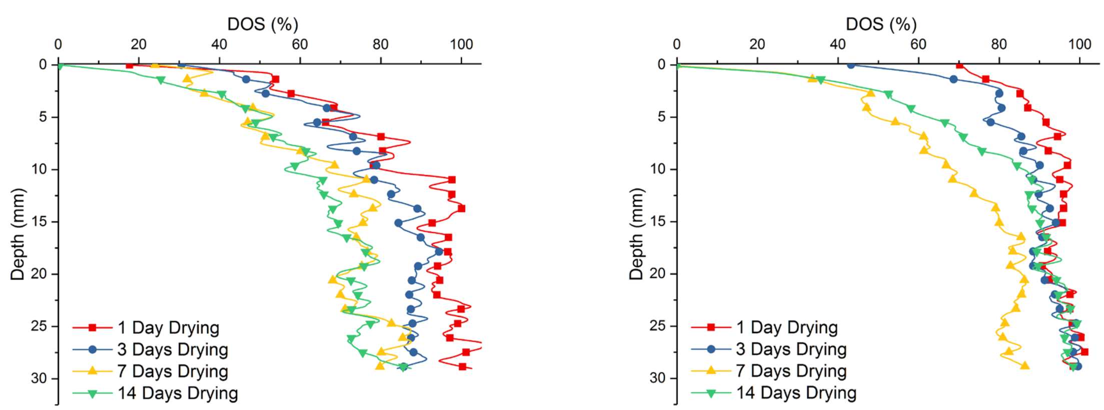
In addition to saturated FLWAs, the possibility of using superabrosbent polymer (SAP) was also investigated in the laboratory (Montanari et al. 2021). One potential benefit of SAPs would be that it would reduce the cost and time for pre-wetting materials as the SAP can be added to the mixer dry and there is no need for stockpile management. This may simplify moisture control at the plant. Research work was performed to develop a method to characterize the superabsorbent polymers and a new testing procedure was proposed. Water absorption of SAP depends on the pH and ionic concentration of the concrete mixture. A slower rate of desorption for IC was also observed when the ionic concentration is increased, for a given relative humidity. As the relative humidity is changed, the SAP releases some of the water, until the solution reaches a salt concentration which is in equilibrium again with the environment. The SAP had an desorption of over 85%, making it suitable for internal curing. Application of SAP to concrete mixtures will require a more rigorous design and testing protocal for a specific mixture. A teabag method has been proposed to assess the SAP absorption (Montanari et al. 2021).

A commercially available SAP was characterized in the lab for use as an internal curing agent in concrete. While many of the measured properties are specific to this particular SAP, the procedures adopted in this lab study can be extended to any SAP used as an internal curing agent. More highly concentrated ionic solutions result in lower absorption. A relatively modest increase in the SAP absorption was observed when comparing the OPC system and the OPC-fly ash systems, e.g., up to 17% with the 60% fly ash replacement. Using SAP that contains 25% of the chemical shrinkage volume resulted in 55% of autogenous shrinkage reduction in the first seven days. This indicates that a reduction on the amount of SAP to include in the mixture is possible while still maintaining a large portion of the benefits. While the mixture proportioning methods have been developed, research is needed to scale this up to field-trials. Properties of concrete mixtures made using SAP need to be evaluated in terms of strength, modulus, transport, and restrained shrinkage.

3.1.3 Internal Curing and Curling

Previous work by Amirkhanian and Roesler (2017) showed IC with FLWA for concrete paving mixtures can reduce the unrestrained curling by as much as 50%. As part of this research study, Montanari et al. (2021) also hypothesized that internal curing should reduce curling of restrained mortar specimens. Neutron radiography was used to measure moisture gradients in real time. It was observed that the mortar samples containing FLWA under self-weight conditions resulted in an increased magnitude of specimen curling relative to the virgin (control) mixture, which was not expected based on the previous findings. The increased in curling is believed to be related to the aggressive drying conditions, i.e., samples were exposed to 50% relative humidity for an extended period of time, as well as the high volume of FLWA and paste in the mortar mixture. Additional testing is currently underway to better understand if this only occurs in aggressive drying (this is believed to be the case due to the size of the pores emptied and the volume of the water lost).

One clear finding with the neutron radiography was the FLWA maintains a higher overall degree of saturation in the specimen under drying conditions compared to a virgin aggregate mixture (See Figure 17), which should improve the near surface quality of the concrete.



(a) (b)

Figure 17. Degree of saturation (DOS) at various stages of drying for the ternary mixture with (a) virgin and (b) FLWA aggregates after 1 day of sealed curing (Montanari et al. 2021).

3.1.4 Internal Curing and Near Surface Curing Effectiveness

The impact of FLWA has been looked at to reduce moisture curling in the previous section. It has been shown (Hajibabaee and Ley 2015; Hajibabaee et al. 2016; Amirkhanian and Roesler 2017) that the presence of extended moist curing can produce significantly higher curling deformations compared to conditions of drying only with minimal additional curing water. Neutron radiography was again used to examine the moisture distribution of specimens with and without FLWA, and substantial differences were observed. Surface moisture content was higher for FLWA mixes. This can improve the surface durability as the hydration process near the surface continues until the degree of saturation is lower than approximately 85%. Furthermore, the FLWA maintains a higher overall degree of saturation in the specimen under drying conditions compared to a virgin aggregate mixture. Beam specimens showed a higher amount of curling for virgin aggregates, when drying was limited to only one surface, as would be the face in pavements.

The influence of curing methods is critical to the hydration of cement in concrete pavements. Neutron radiography also was used to quantify the extent of hydration at various distances from the drying surface. This is particularly useful in determining the 'curing affected zone' (CAZ). In the mixture exposed to drying after 1 day, the top 12.5 mm (1/2 inch) of the mortar was dramatically impacted by the loss of water to evaporation. While the top 5 mm of the surface of the sample exposed to drying at 1-day had a degree of hydration that was 32% less than the 14-day moist cured sample. As such, the duration of curing is important, especially with supplementary cementitious materials. The use of IC (e.g., SAP) demonstrated benefits in terms of both an increase in the overall degree of hydration (this can conservatively be assumed to be approximately 3.7 to 7.8%) and a reduction in the depth of the CAZ by 4 mm (1/6 in) when SAP is used. Finally, IC creates a more well hydrated and durable surface, which is a significant benefit that can be quantified now in terms of CAZ.

3.2 MACRO-FIBERS IN CRCP

The potential addition of synthetic macro-fibers to CRCP has multiple purposes. Macro-fibers have been demonstrated to increase the structural capacity of concrete slabs for industrial floor systems, jointed concrete pavements, and concrete overlays. In addition, macro-fibers extend the service life of cracked slabs by reducing the rate of crack deterioration through crack widith reduction. Macro-fibers have also shown to keep steel reinforced concrete crack widths to a lower magnitude. Therefore the main expected benefit of macro-fibers in CRCP is to reduce the steel content and still keep the crack widths much below the design threshold of 0.5mm for CRCP. The design features for each ATREL and Tollway field section is summarized in Figure 4 (Chapter 2) and Tables 4 (Chapter 2). Tables 5, 6, and 7 below demonstrates for the same steel content how macro-fibers had similar or tighter crack widths for both the continuously reinforced concrete beams (CRCB) at UIUC (sections 2A and 2D) and the Illinois CRCP test sections (sections 1D and 3A), respectively.

Table 5. Average Crack Width with Standard Deviation for Sections 1 and 2 of ATREL Continuously Reinforced Concrete Beam on Day 917

Section	CW (mm)	Standard Deviation (mm)
1A	0.1	0.01
1B	0.1	0.00
1C	0.13	0.04
1D	0.13	0.03
2A*	0.13	0.05
2B	0.15	0.06
2C	0.12	0.03
2D*	0.11	0.02

*Macro-fiber added to concrete mixture

Table 6. Average Crack Width (mm) and Standard Deviation at Different Ages for Section 1 with Air Temperature

Section	Days			
	80		640	
	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)
1A	0.59	0.25	0.41	0.13
1B	0.57	0.13	0.55	0.18
1C	0.64	0.20	0.79	0.27
1D	**0.39**	**0.19**	**0.55**	**0.22**
1E	0.50	0.16	0.65	0.25
Temperature (F)	45		77	

Table 7. Average Crack Width (mm) for Section 3 at Different Ages with Air Temperatures

Section	Days			
	65		375	
	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)
3A	**0.11**	**0.02**	**0.16**	**0.06**
3B	0.16	0.07	0.16	0.08
Temperature (F)	80		81	

3.3 CORROSION RISK FOR BLACK STEEL WITHOUT EPOXY-COATING

Transverse cracks will accelerate the ingress of the chlorides in reinforced concrete. However, the transverse cracks in CRCP are relatively small and the reinforcing steel surface cover is substantially deeper than in other concrete structural sections. This results in the transverse cracks being relatively tight with depth and making the chlorides transport through the cracks difficult. In a discussion with both ARA, Inc and Tollway engineers during the project, it has been reported that the field corrosion of the reinforcing steel is not a significant issue that is limiting the life of CRCP on Tollway assets. If the epoxy-coated steel was not required, substantial reductions in the initial cost will be realized assuming no reduction in performance life.

In order to verify this hypothesis, concrete samples from existing Illinois Tollway CRCP sections (I-80) were analyzed to study chloride ingress and to assess whether the existing designs are prone to chloride ingress that may damage the steel, which is positioned 3.5 inches from the surface. From the analyzed core samples, it was found that chloride ingress did not reach the depth of the steel even at the transverse crack. Over time, chloride concentrations will continue to increase at the transverse cracks in the CRCP and thus wide transverse cracks are the main threat to non-epoxy-coated steel. Given the initial field evidence suggesting that epoxy coating of the steel would not be necessary because of a deeper cover and tight crack widths, the Tollway constructed the field test sections and UIUC constructed the ATREL reinforced concrete beam sections without epoxy coated steel (black bar).

3.4 SUPPLEMENTARY CEMENTITIOUS MATERIALS WITH INTERNAL CURING

The use of supplementary cementitious materials was investigated for both internal curing and its role in helping to reduce initial thermal gradients near the time of final concrete setting. Mixtures were able to be successfully made with supplementary cementitious materials and the use of supplementary cementitious materials reduces potentially deleterious reactions with deicing salts. It is recommended that the Illinois Tollway continues to utilize supplementary cementitious and admixtures because they provide short-term and long-term benefits.

3.5 CONCRETE FREEZE-THAW DURABILITY AND DISTRESS

Distress has recently been observed in some concrete pavements throughout the Midwest in wet-freeze states, primarily at the joints in JPCP. This distress often begins in longitudinal joints, followed by transverse joints and results in the significant loss of material from the joint area. This deterioration greatly reduces the service life and increases maintenance costs of the pavements. Early in this project, it was determined that this joint durability issue was not being significantly observed in CRCP. However, steps and design changes being developed for JPCP should be considered in the CRCP mixtures. The following list provides an excerpt of the main findings and recommendations for INDOT (SPR 3808) followed by implications for the Illinois Tollway future design and construction of CRCP or JPCP (Montanari et al. 2021).

1) Increase the specified volume of air entrainment and reduce the variation in air content. The recommendation for the Tollway is to continue to require a total volume of air of 6.5% and to encourage contractors to reduce the variation through the quality control processes.
2) Reduce the volume of cementitious paste in concrete pavements. The recommendation for the Tollway is to encourage mixtures that have optimized aggregate gradations that enable the paste volume to be reduced below 25%. Any minimum cement contents should be considered to be removed from the specification.
3) Reduce the water-to-cementitious materials used in concrete pavements. As it is related to the Tollway, it is suggested that the maximum water to binder (w/b) ratio be limited to 0.42.

4) Use of a formation factor to specify the transport properties of concrete. This recommendation is not ready to be implemented at the current time with the Tollway. However, it should be considered in the future in coordination with AASHTO PP-84.

5) Use supplementary cementitious materials (SCM) to reduce susceptibility to salt damage/ use a performance test for mixture design to limit calcium oxychloride damage. This recommendation is not ready to be implemented at the current time with the Tollway.

6) Use a topical treatment for concrete that repels water or seals the concrete. This recommendation is not ready to be implemented at the current time with the Tollway.

7) Use a capillarity break below the pavement. This recommendation suggests the use of a more drainable subbase layer below the pavement, e.g., PGE subbase layer. This should be considered with the complete pavement section design.

8) Reduce the strength required to open a pavement to traffic. This recommendation is not ready to be implemented at the current time with the Tollway.

9) Improve the use of methods to detect water ponding in concrete pavements. This recommendation is not ready to be implemented at the current time with the Tollway.

10) Examine the proportion of salts in blended systems. This recommendation refers to deicing practice rather than mixture proportioning or pavement design.

CHAPTER 4: FOUNDATION MATERIALS

The performance of CRCP is strongly linked to the non-erodibility of the base/subbase layers (Jung et al 2009, 2010, 2012). In order to improve the economics of CRCP design, the base layer has to be designed as a composite layer with the CRCP slab, which means knowledge of the long-term frictional bond between the slab-base is required. Secondly, the in-situ structural capacity needs to be verified for sections with non-erodible bases which can be done for structural capacity of the CRCP test sections on Illinois Route 390. Additionally, the cost effectiveness of the overall design is linked to how the exisiting pavement structure will be used and processed into the new pavement structure. This means an base/subbase erodibility performance test is required in order to evaluate a combination of base/subbase materials that can utilized recycled materials and cement/asphalt binder.

4.1 BASE LAYER DESIGN

In order to provide for a cost effective CRCP design, the base layer must be provide maximum structural support to the CRCP slab, be non-erodible (Jung et al. 2012), be constructed out of materials from the existing roadway, and potentially be constructed with minimal movement of the existing pavement structure to off site facilites. Asphalt concrete base layers have performed extremely well for CRCP designs in the state of Illinois. Given the volume of materials available for base and subbae layers, a performance specification must be developed that assesses potential combinations of recycled and virgin materials along with asphalt and cement binders.

The key base design factors consist of the following: (1) Thickness, (2) Modulus, (3) Shear strength, and (4) angle of repose.The first two factors affect the structure stiffness of the base layer and the composite support provided to the concrete slab while the last two factor affect the strength of the bond with an adjacent layer like the CRCP surface layer. All four of these properties pertain to the performance of the base layer regardless of the nature of the stabilization. Base performance is defined with respect to the degree of bond the base layer affords the pavement slab and its resistance to consolidation effects under normal and confining stress levels which is one of the modes of erosive-related behavior that causes an increase in bending stress and loss of fatigue life in the concrete pavement. The combination and thicknesses of the foundation layers is a balance between the available material, level of erodibility, required frictional resistance and bending stiffness, and necessity for drainage to avoid pore water pressure development and water jetting.

4.2 SLAB-BASE FRICTIONAL BOND

The interface condition between the slab-base for the same type of material, cementitious or asphaltic, is important to the long-term performance of the base layer and it provides for enhancing the structural capacity of the CRCP slab through composite action. The assumption that a cement or asphalt stabilized base layer is fully bonded or unbonded has not been supported by field concrete pavement data (Rufino and Roesler 2006; Khazanovich and Tompkins 2017). The slab-base interface may be considered bonded but in many cases there is a frictional bond that provides composite action with the CRCP slab beyond an unbonded concrete interface assumption but in most instances, not quite a fully bonded assumption either. The frictional coefficient between the slab-base interface was characterized for the Tollway test sections on Illinois Route 390. Figure 18 shows the backcalculated non-dimensional frictional coefficient for section 1 with higher values suggesting more bonded interface and lower values approaching unbonded interface. The values presented suggest that the microsurfacing and WMA interlayers both provided signficant frictional resistance such that the composite action of the CRCP plus layer was acting as a much thicker equivalent layer as seen in Table 8. The nominal design thickness of section 1 was 10.5 inches, which suggests a significant composite action occurring with the stabilized base layer and asphalt interlayer.

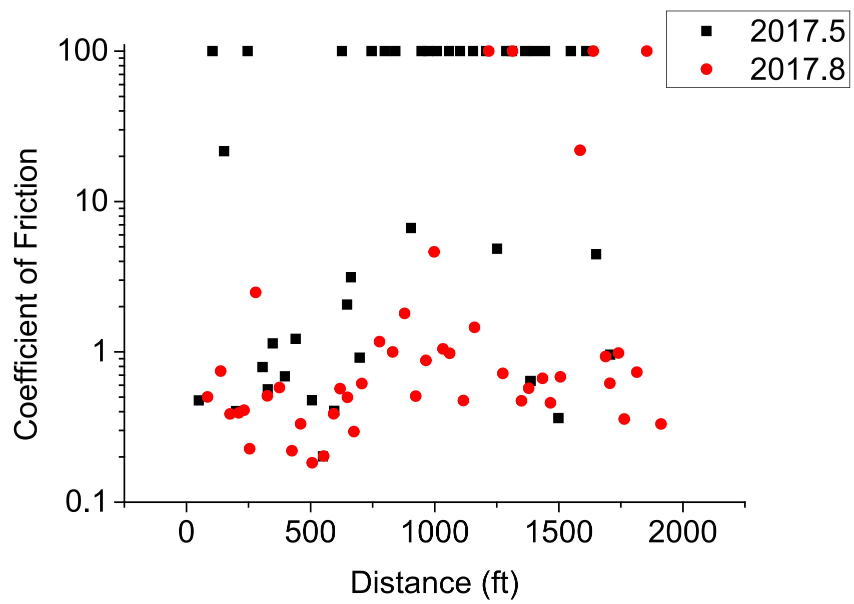
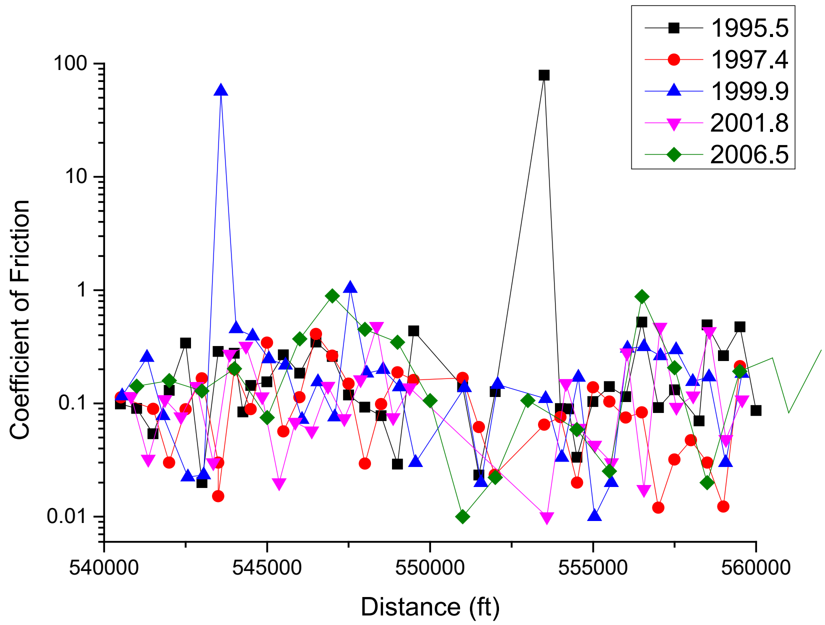


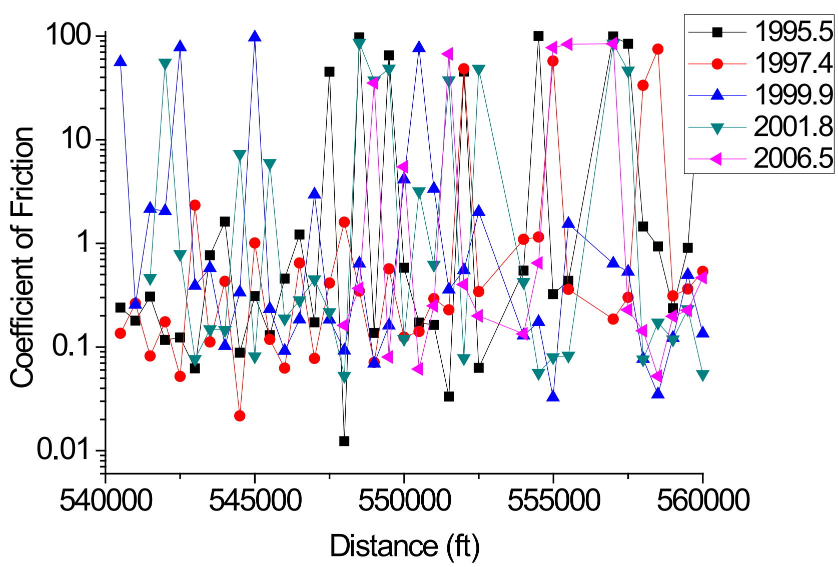
Figure 18. Non-dimensional coefficient of friction along test section 1 from two FWD tests.

Table 8. Summary of Effective Slab Thickness and Non-Dimensional Friction Coefficient (Λ*) for Tollway Test Section 1

Test section	FWD test on May 2017		FWD test on August 2017	
	h_{eff} (in.)	Λ*	h_{eff} (in.)	Λ*
1E	13.5	1.51	12.7	0.46
1D	13.5	1.73	12.6	0.39
1C	14.5	100	13.3	0.99
1B	14.6	100	13.5	1.15
1A	14.0	7.49	13.4	0.91



(a) Northbound



(b) Southbound

Figure 19. Non-dimensional coefficient of friction over time in (a) northbound and (b) southbound in the CRCP test section on I-57 from years 9 to 20.

The next question regarding the interface is how long will the interface frictional resistance measured during the first few years exist in the long-term. To answer this question, Figure 19 presents an analysis completed on I-57 CRCP with cement-stabilized base layer from year of service ranging 9 to 20. Table 9 lists the average coefficient of friction, effective thickness, and effective stiffness on both southbound and northbound CRCP test sections from 1995 to 2006. Table 7 indicates no significant variation of friction coefficient over time. There are two possible reasons for this relative stable coefficient of friction. First, the in-place cement stabilized base has provided good, stable support to the CRCP slab without much erosion. This is indicated by the stable coefficent of friction over time. Secondly, the first FWD test in this test section was in 1995, 9 years after the construction and as a result, it is possible that the interface condition between the slab and base had dropped to a steady-state value but was different than the initial values that are seen on the Tollway sections. However, the steady coefficient of friction 9 years after construction indicates that the cement-stabilized base was providing good support to CRCP slab, which is indicated also by the overall positive pavement performance (Roesler et al 2011). This test section was eventually overlaid with asphalt concrete because of spalling of longitudinal cracks on the surface that had developed over time.

Table 9. Average Effective Slab Thickness (h_{eff}) and Non-Dimensional Friction Coefficient (Λ^*) for I-57 over Time with FWD Tests

Time of FWD tests	Northbound		Southbound	
	h_{eff} (in.)	Λ^*	h_{eff} (in.)	Λ^*
1995.5	11.0	0.17	11.83	0.53
1997.4	10.4	0.09	11.59	0.35
1999.9	10.7	0.14	11.67	0.38
2001.8	10.5	0.11	11.88	0.55
2006.5	10.9	0.17	11.90	0.55

The results show a stable interface friction from a non-erodible base layer can be assumed to be part of the structural resistance of the system for a Tollway CRCP system.

4.3 BASE EROSION

The key to designing the foundation layers is making sure the combination of materials is non-erodible and offers frictional resistance both for crack development and composite action of the CRCP slab. Volume 5 of this research report provides the background and procedure for evaluating the erosion resistance of the subbase layer (Speakmon et al. 2021) with the Hamburg Wheel Tracker testing device.

4.4 SUBGRADE

The natural support conditions have been modified and/or the appropriate lower quality granular material has been placed to lower the stress state on the natural occurring soil in the corridor of Illinois Rt 390. FWD testing has shown the composite k-value of all material below the cement-treated base layer (See Figure 20) is quite sufficent for long-term performance, i.e., much greater than the natural soil of 50 to 100 psi/in k-value. The average k-value was 455 psi/in based on the new finite-sized panel CRCP backcalculation procedure developed in report volume 3 by Zhang and Roesler (2021).

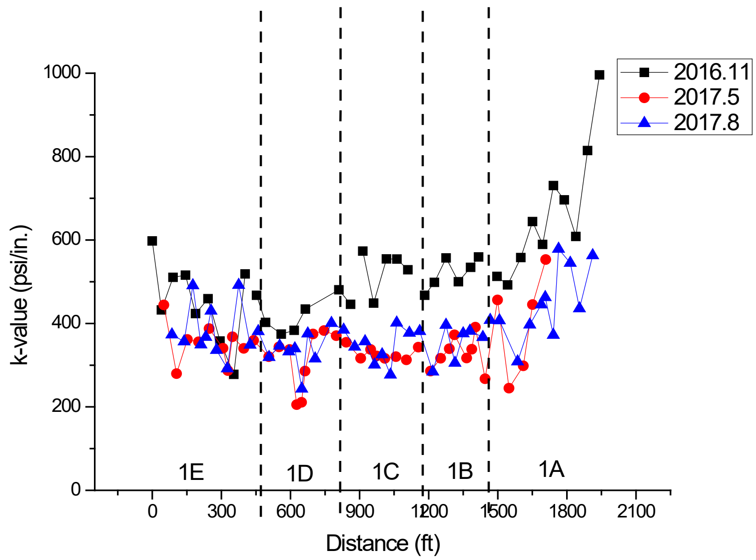


Figure 20. Backcalculated *k*-value in Tollway section 1.

CHAPTER 5: CRCP STRUCTURAL DESIGN IMPROVEMENTS

CRCPs are structurally designed to naturally develop a pattern of transverse cracks at regular intervals, and these cracks serve as contraction joints. The intended purpose of these controlled cracks are to possess a high load transfer and stress relief characteristics that minimize the potential for fault or spall development and eventual loss of ride quality. Due to the features of the cracking pattern, CRCPs have a high potential to maintain a relatively low state of tenile stress under mechanical and environmental loading and provide excellent levels of performance as a result.

5.1 CRACK SPACING AND WIDTH

The ideal crack pattern is one that is uniformly distributed and initiated early in CRCP life, which should lead to cracks that have similar but smaller movements. Many of the initial cracks in CRCP form shortly after construction and the majority of the final cracking pattern within the first 1 to 2 years especially after one to two winter (contraction) cycles. The magnitude of crack opening and closing has been recognized for many years as a key component of CRCP since if affects structural stiffness of the transverse crack, moisture infiltration to the interface of the slab/subbase contact, and minimizes deleterious substances such as chlorides to attack the steel.

The factors that affect crack movement are linked to the following:

- Spacing between cracks,
- Season of placement,
- Percent and depth of steel reinforcement, and
- Subbase type and interface frictional resistance.

The capability of a CRCP systems to maintain acceptable crack movement limits depends upon the balance between each of the above bulleted items and the timing and the uniformity of the initiation of the cracking. Random cracking patterns tend to be distributed over a range of 3 to 8 feet but spacing wider than 8 feet are common. Two conditions that are related to this balance is the development of early cracks and the lack of critical stress development in the vicinity of terminal or header joints in CRCP. Random developing early cracks tend to be widely spaced and manifest wider movements than those that form later. Cracks that form in the vicinity of terminal joints also tend to be widely spaced but exhibit smaller movements because the end of the terminal joint absorbs a large portion of the slab movement. One of the findings of this study is the need to maintain a better balance in design between the above factors and different sections of a CRCP to more effectively affect the development of the characteristics of an ideal crack pattern.

Crack spacing was measured using a distance measuring wheel. The CRCB sections at ATREL that were non-internally cured (1C and 2C) developed a smaller crack spacing especially at early-ages compared to similar sections with internal curing (1B and 2B) as seen in Tables 10 and 11. In general, the CRCB section 1 with higher steel content was found to have smaller crack spacing (and tighter crack widths) compared to CRCB section 2 with lower steel content. Several lower steel sections, 2B and 2C, had consistently higher crack spacing (and wider crack widths) relative to the higher steel section (1B) as seen from Tables 10 and 11. For the IC with higher steel content the crack spacing (and width) were smaller than the virgin concrete mixture. For both CRCB sections, non-internally cured section had the lowest crack spacing overall at 917 days.

Table 10. Average Crack Spacing with Standard Deviation for Section 1 of ATREL CRCB

Days	Section 1A		Section 1B		Section 1C		Section 1D	
	Avg CS (ft)	Standard Deviation (ft)	Avg CS (ft)	Standard Deviation (ft)	Avg CS (ft)	Standard Deviation (ft)	Avg CS (ft)	Standard Deviation (ft)
6	N/A	N/A	46.0	0.0	34.8	22.1	17.3	20.6
14	N/A	N/A	27.0	15.5	10.7	5.7	8.4	3.2
49	8.0	4.7	6.4	4.2	5.0	1.8	5.7	2.3
79	7.9	3.6	5.0	3.1	4.0	1.6	5.4	2.4
182	6.2	2.4	4.5	2.3	3.5	1.8	4.6	2.5
300	6.2	2.1	4.0	2.5	3.5	1.7	4.0	1.9
569	4.7	2.2	3.1	2.4	2.4	1.4	2.9	1.4
917	3.2	1.5	2.5	1.5	2.3	1.2	2.9	1.6

Note: No cracks were formed in section 1A until day 14.

Table 11. Average Crack Spacing with Standard Deviation for Section 2 of ATREL CRCB

Days	Section 2A		Section 2B		Section 2C		Section 2D	
	Avg CS (ft)	Standard Deviation (ft)	Avg CS (ft)	Standard Deviation (ft)	Avg CS (ft)	Standard Deviation (ft)	Avg CS (ft)	Standard Deviation (ft)
6	N/A	N/A	22.3	8.3	10.8	11.7	13.1	6.9
14	17.7	7.0	22.3	8.3	10.7	11.3	8.7	5.5
49	6.6	3.2	8.8	4.1	4.8	3.1	4.1	1.9
79	5.9	3.0	6.4	3.1	4.3	2.8	4.9	2.5
182	5.8	3.8	5.8	2.6	4.0	2.7	4.2	2.1
300	5.5	3.2	6.2	2.3	3.7	1.9	4.0	2.1
569	4.0	2.1	3.8	1.7	2.9	1.4	3.6	1.5
917	3.8	1.8	4.1	1.7	2.7	1.2	3.6	1.8

For the Illinois Tollway sections 1 and 2 in Tables 12 and 13, higher steel content sections (1A, 1D, 1E, and 2A) have lower average crack spacing than the lower steel content sections (1B,1C, and 2B), as expected. From Figure 21, the average crack spacing after approximately 100 days for Tollway section 1 with internal curing is consistently higher than that of Tollway section 2 without internal curing. The average crack spacing of Tollway section 3 continues to decrease in Figure 21 because of activation of a large number of cracks through the partial width/depth notching procedure (Table 14).

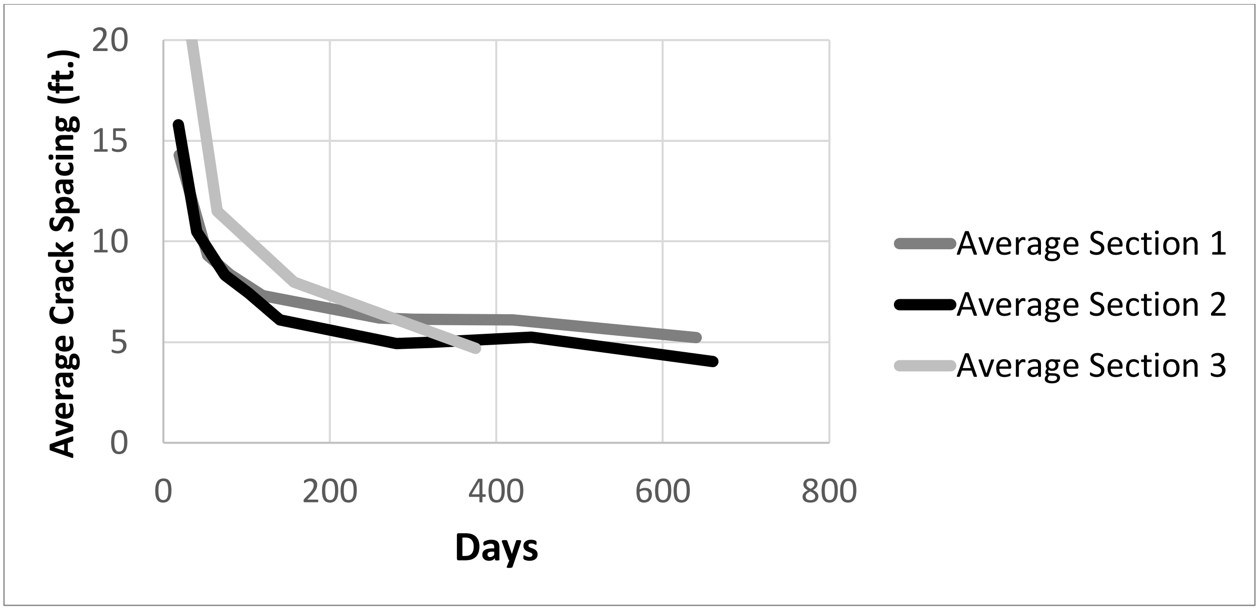


Figure 21. Average crack spacing of all three sections at different age.

CRCP field test Section 2 (non-IC) has distribution of crack spacing that is skewed more towards left indicating higher number of lower crack spacing on day 323 as seen in Figure 22 with about 43% crack spacing lower than 3 ft. compared to about 31% in Section 1 (with IC). Cracks less than 3 ft are near the lower desirable range of crack spacing and development of any further cracks in that section would increase probability of cluster, divided, or y-crack formation, which is called here undesirable cracks. In section 3, with active cracking, only 8% of the cracks are smaller than 3 ft. on day 375 indicating a lower probability of forming undesirable cracks. Similar observations have been made in Europe as well for sections with active crack control (Ren 2015; Ren et al. 2014).

CRCP test sections 1 and 2 crack spacing distribution for day 640 and day 660, respectively, are presented in Figure 23. After almost 2 years, 57% of crack in section 2 (non-IC) are smaller than 3 ft while only 40% of cracks in section 1 are smaller than 3 ft. Likewise, 43% of transverse crack spacing is less than 2 ft. in section 2, which increases likelihood of future undesirable cracks. There is an improvement in section 1 in reducing the number of cracks less than 2 ft (only 28%). However, internal curing has not eliminated all undesirable crack spacings, which requires additional construction processes and will be discussed later in this section.

Table 12. Internally Cured Tollway CRCP Test Section 1 Average Crack Spacing (ft.) and Standard Deviation (ft.) at Various Days after Construction

Section	Days															
	19		53		80		119		259		300		420		640	
	CS (ft.)	Std Dev (ft.)	CS (ft.)	Std Dev (ft.)	CS (ft.)	Std Dev (ft.)	CS (ft.)	Std Dev (ft.)	CS (ft.)	Std Dev (ft.)	CS (ft.)	Std Dev (ft.)	CS (ft.)	Std Dev (ft.)	CS (ft.)	Std Dev (ft.)
1A	14.0	5.7	10.2	3.0	9.7	3.3	8.2	4.5	7.2	4.4	7.4	4.5	6.9	4.5	5.2	3.0
1B	16.6	8.7	9.1	3.7	8.8	3.5	7.7	3.9	6.6	4.0	6.4	3.2	6.6	3.0	6.0	3.0
1C	16.4	8.4	10.6	3.4	9.0	3.6	8.4	3.5	7.3	3.3	7.1	3.4	7.2	3.4	6.1	3.7
1D	11.7	6.2	8.2	4.2	6.6	3.6	5.0	3.2	4.0	2.7	3.9	2.7	3.9	2.7	3.3	2.4
1E	12.9	7.0	8.5	3.5	7.7	3.8	7.4	5.3	5.9	4.0	5.9	4.0	6.0	3.9	5.6	3.9

Table 13. Not-Internally Cured Tollway CRCP Test Section 2 Average Crack Spacing (ft.) and Standard Deviation (ft.) at Various Days after Construction

Section	Days																	
	18		40		74		102		140		280		323		442		660	
	CS (ft.)	Std Dev (ft.)	CS (ft.)	Std Dev (ft.)	CS (ft.)	Std Dev (ft.)	CS (ft.)	Std Dev (ft.)	CS (ft.)	Std Dev (ft.)	CS (ft.)	Std Dev (ft.)	CS (ft.)	Std Dev (ft.)	CS (ft.)	Std Dev (ft.)	CS (ft.)	Std Dev (ft.)
2A	15.0	6.6	10.9	5.4	8.0	4.8	6.9	4.0	5.9	3.5	4.4	3.0	4.4	2.7	4.7	4.7	3.7	4.2
2B	16.6	6.7	10.1	5.7	8.7	4.1	7.9	4.2	6.4	3.5	5.5	3.4	5.6	3.4	5.8	4.7	4.4	3.4

Table 14. Internally Cured Tollway CRCP Test Section 3 Average Crack Spacing (ft.) and Standard Deviation (ft.) at Various Days after Construction

Section	Days							
	22		65		157		375	
	CS (ft.)	Std Dev (ft.)	CS (ft.)	Std Dev (ft.)	CS (ft.)	Std Dev (ft.)	CS (ft.)	Std Dev (ft.)
3A	32.2	14.5	11.4	5.4	7.6	4.2	5.0	2.1
3B	14.5	3.7	11.6	8.3	8.3	4.4	4.4	2.2

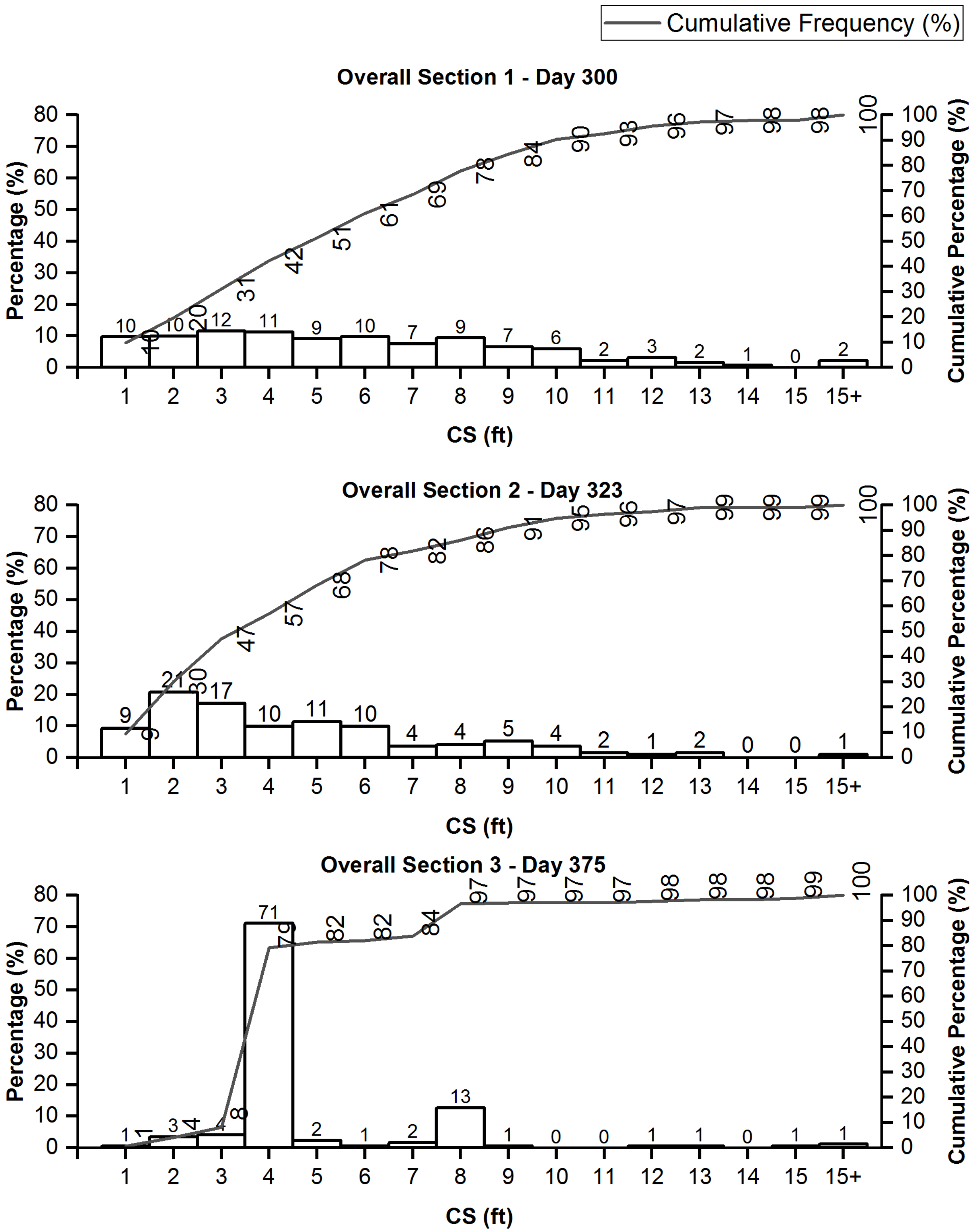


Figure 22. Crack spacing distribution for three CRCP test sections on similar age (between 300 and 375 days).

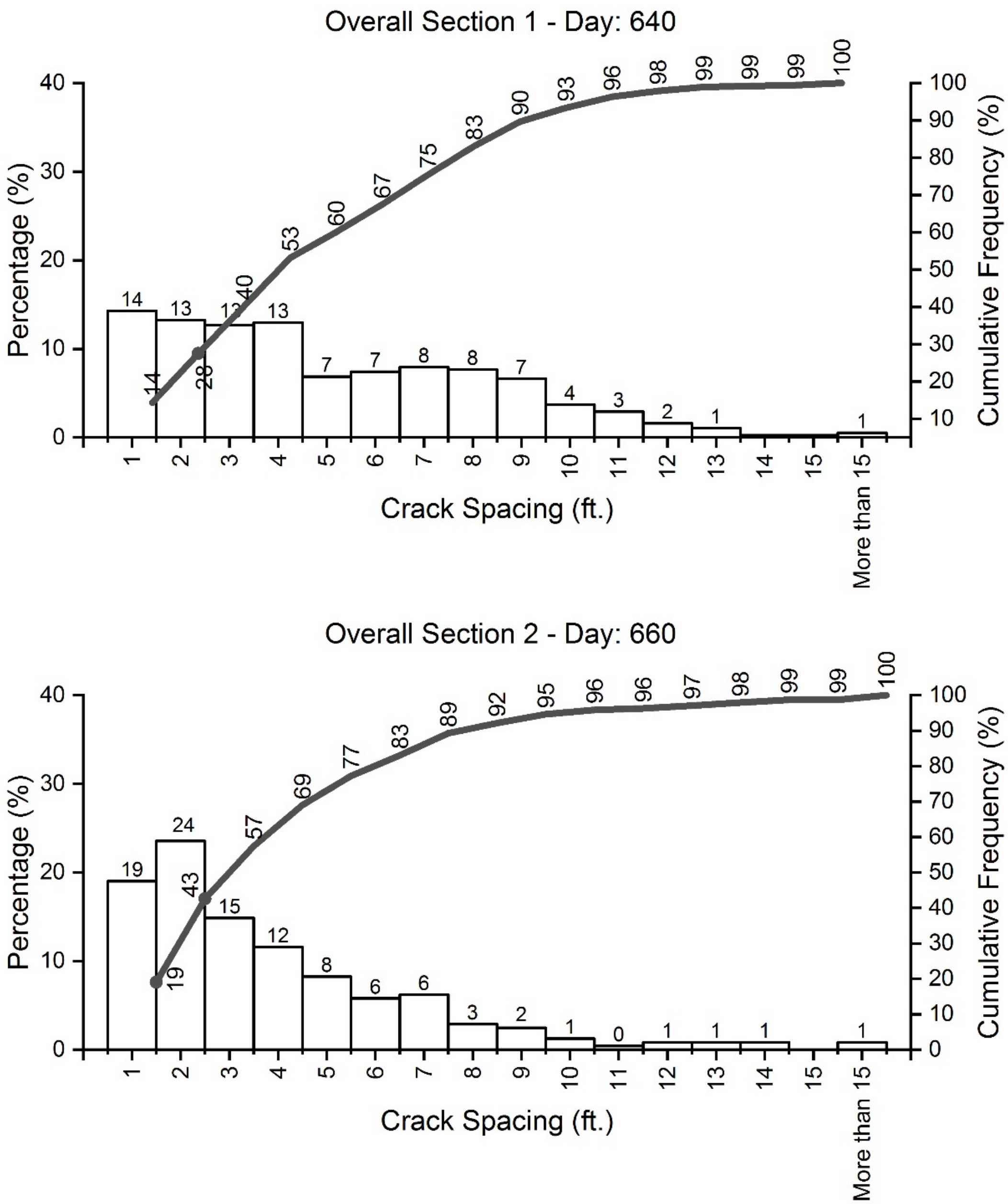


Figure 23. Crack Spacing of overall sections 1 and 2 on day 640 and day 660, respectively.

Crack widths were measured on all sections over time with the manual comparator shown in Figure 24. In the CRCBs, Section 1 had a smaller crack width as a result of the higher steel content versus section 2 with lower steel. The effect of macro-fibers could be observed in Sections 2A and 2D in general having slightly lower crack width compared to 2B and 2C as seen in Table 15. In addition, section with lower steel and macro-fiber had comparable crack width as higher steel sections with and without IC.

Figure 24. Crack width comparator for surface crack width measurement.

Table 15. Average Crack Width with Standard Deviation for Sections 1 and 2 of ATREL CRCB

Days	Section 1A		Section 1B		Section 1C		Section 1D	
	Avg CW (mm)	Standard Deviation (mm)	Avg CW (mm)	Standard Deviation (mm)	Avg CW (mm)	Standard Deviation (mm)	Avg CW (mm)	Standard Deviation (mm)
300	0.10	0.00	0.10	0.00	0.10	0.00	0.10	0.00
569	0.11	0.02	0.10	0.00	0.11	0.03	0.11	0.03
917	0.10	0.01	0.10	0.00	0.13	0.04	0.13	0.03
Days	Section 2A		Section 2B		Section 2C		Section 2D	
	Avg CW (mm)	Standard Deviation (mm)	Avg CW (mm)	Standard Deviation (mm)	Avg CW (mm)	Standard Deviation (mm)	Avg CW (mm)	Standard Deviation (mm)
300	0.10	0.00	0.10	0.00	0.10	0.00	0.10	0.00
569	0.12	0.04	0.23	0.10	0.14	0.06	0.11	0.03
917	0.13	0.05	0.15	0.06	0.12	0.03	0.11	0.02

The crack widths over time measured for the Tollway field sections 1 to 3 are shown in Tables 16 to 18. As seen in the Figure 26 from the crack width distribution, section 3 has the higher percentage of smaller crack widths. Section 3 was constructed on a cooler temperature condition while the other two sections had much higher paving temperatures. For section 1, about 40% of cracks are smaller than 0.4 mm (see Table 16 and Figure 26), while section 2 had about 47% cracks smaller than 0.4 mm at the age around day 300 (see Table 17 and Figure 26). Section 3 has 100% of the crack width smaller than 0.4 mm and more than 90% smaller than 0.3 mm even after day 375 (see Table 18 and Figure 26). Several reason led to this performance in section 3 including it was constructed at much lower temperature and cracks were initiated earlier with active notches relative to sections 1 and 2.

Overall, crack width were dependent on temperature during the time of survey but still was observed to increase over time as seen in Tables 16 to 18. For similar age and temperature, the IC section 1 consistently had smaller crack width compared to the non-IC section 2 as demonstrated in Tables 16 and 17 as well as Figure 26 and 27. Statistical analysis, as discussed in volume 2 (Dahal et al. 2021), showed that the crack width of these two sections were statistically different for similar age and temperature. Section 3 had significantly lower crack width than other two sections at observations made after only one year.

Figure 25 shows the percent decrease of the average crack width for the IC section compared to the non-internal cured section (Section 1 versus Section 2). It is clear that the IC sections crack width is always smaller than the non-IC. The variation of crack width did depend on the temperature, however, IC section average crack width were around 10 to 45% smaller than the NIC section average crack widths. On average, the IC sections crack widths were approximately 25% lower than the Non-IC sections crack width.

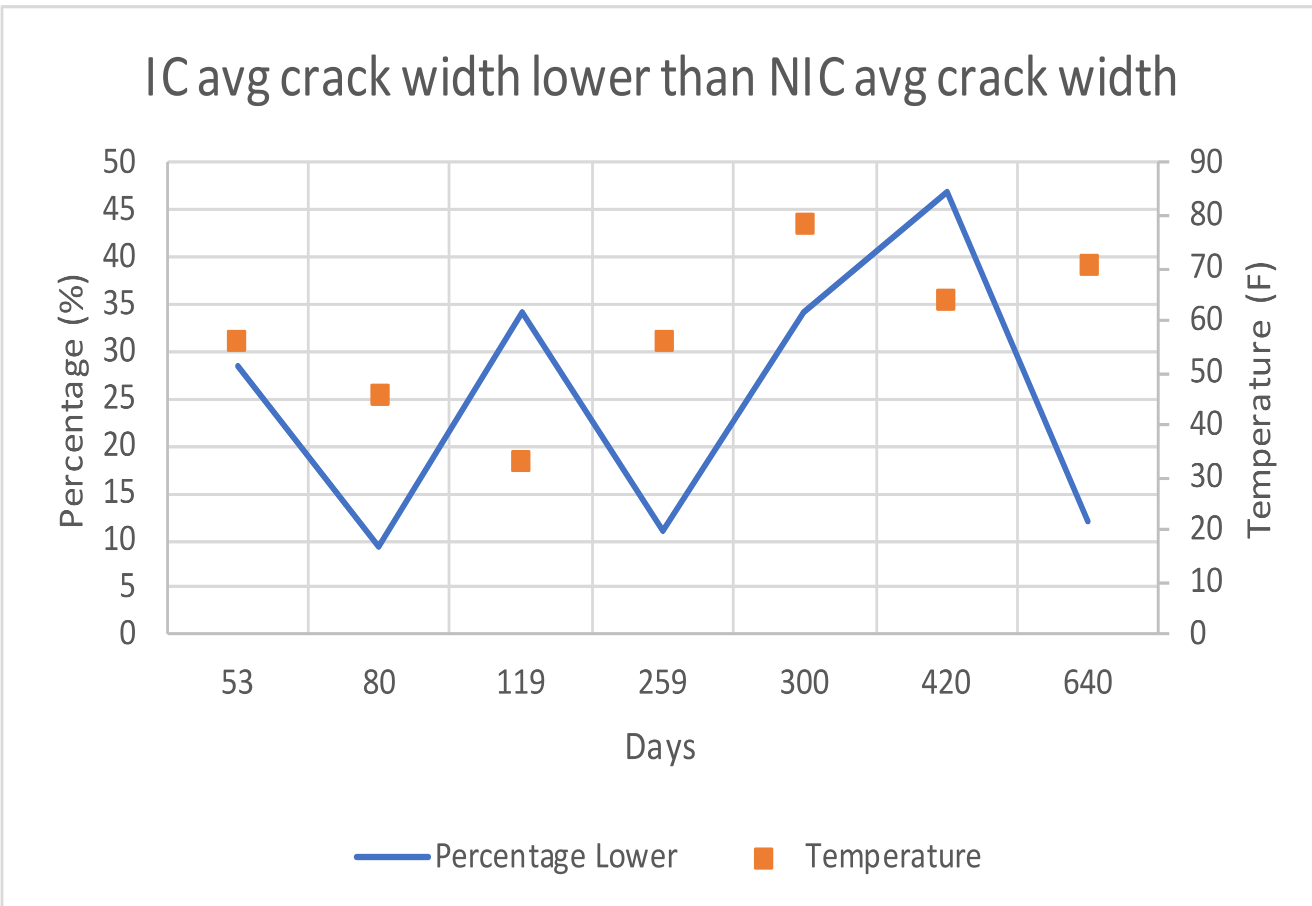


Figure 25. Percent decrease in average crack width from internally-cured section relative to non-internally-cured concrete section.

Table 16. Average Crack Width (mm) and Standard Deviation at Different Ages for Section 1 with Air Temperature

Section	Days															
	19		53		80			119	259		300		420		640	
	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)
1A	0.21	0.03	0.38	0.17	0.59	0.25	0.38	0.16	0.36	0.16	0.44	0.18	0.54	0.16	0.41	0.13
1B	0.20	0.00	0.31	0.12	0.57	0.13	0.40	0.15	0.55	0.21	0.53	0.22	0.60	0.17	0.55	0.18
1C	0.21	0.03	0.45	0.16	0.64	0.20	0.47	0.17	0.56	0.22	0.60	0.22	0.60	0.19	0.79	0.27
1D	0.21	0.04	0.28	0.16	0.39	0.19	0.27	0.10	0.36	0.15	0.38	0.22	0.38	0.12	0.55	0.22
1E	0.24	0.05	0.31	0.09	0.50	0.16	0.36	0.12	0.47	0.20	0.38	0.17	0.45	0.16	0.65	0.25
Mean of Section	0.21	0.03	0.35	0.14	0.54	0.19	0.38	0.14	0.46	0.19	0.47	0.20	0.51	0.16	0.59	0.21
Temperature (F)	N/A		55		45			35	55		76		64		77	

Table 17. Average Crack Width (mm) and Standard Deviation at Different Age for Section 2 with Air Temperature

Section	Days																	
	18		40		74		102		140		280		323		442		660	
	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)
2A	0.26	0.06	0.42	0.12	0.50	0.19	0.51	0.22	0.44	0.17	0.44	0.25	0.64	0.25	0.69	0.24	0.63	0.29
2B	0.24	0.06	0.40	0.12	0.40	0.17	0.67	0.30	0.57	0.25	0.58	0.28	0.62	0.21	0.80	0.31	0.70	0.29
Mean of Section	0.25	0.06	0.41	0.12	0.45	0.18	0.59	0.26	0.51	0.21	0.51	0.27	0.63	0.23	0.75	0.27	0.66	0.29
Temperature (F)	95		N/A		58		47		32		58		82		64		65	

Table 18. Average Crack Width (mm) for Section 3 at Different Ages with Air Temperatures

Section	Days							
	22		65		157		375	
	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)	CW (mm.)	Std Dev (mm.)
3A	0.13	0.05	0.11	0.02	0.20	0.07	0.16	0.06
3B	0.21	0.11	0.16	0.07	0.16	0.06	0.16	0.08
Mean of Section	0.17	0.08	0.13	0.05	0.18	0.07	0.16	0.07
Temperature (F)	46		80		64		81	

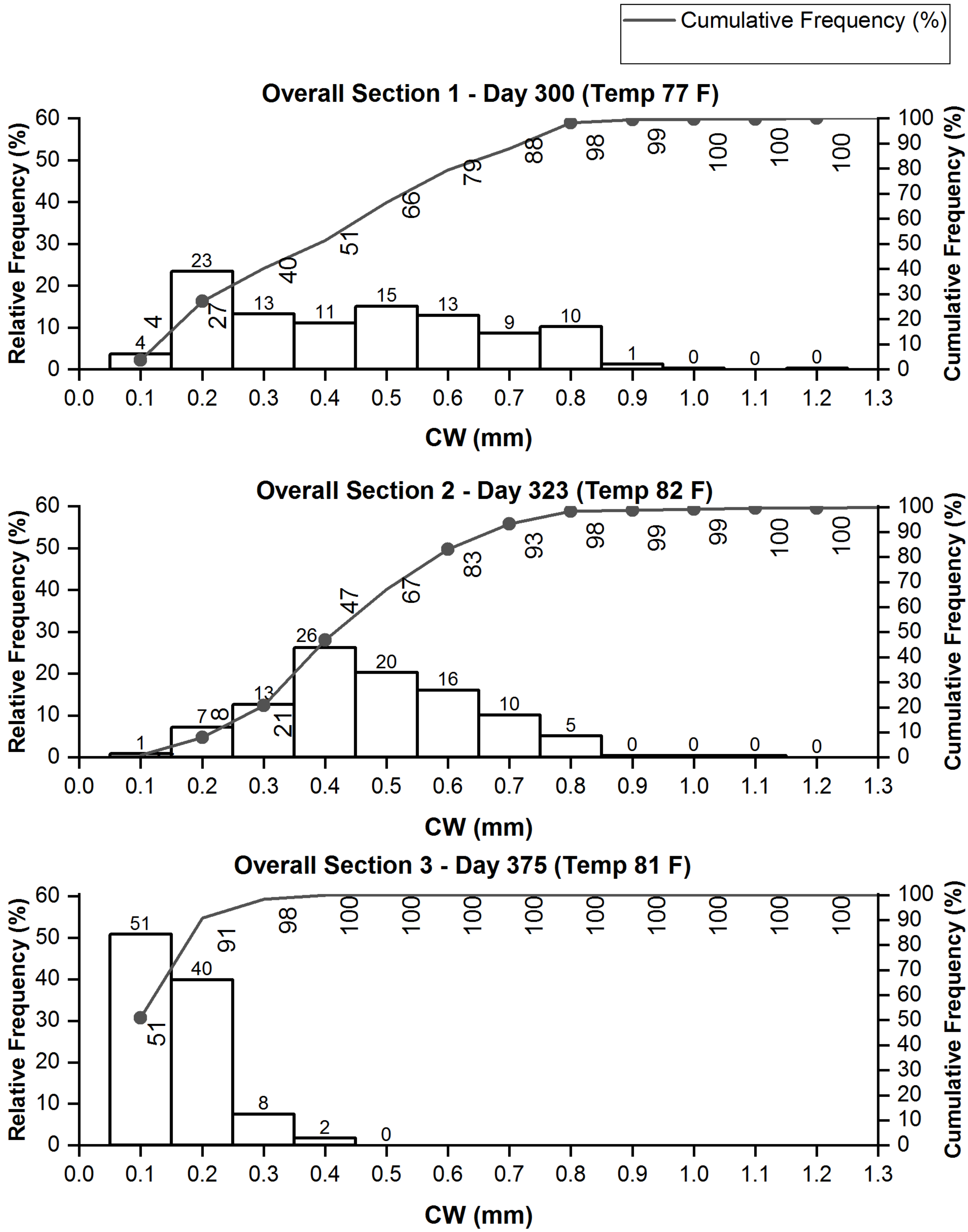


Figure 26. Comparison of crack width for section 1, section 2, and section 3 after 300 days of construction.

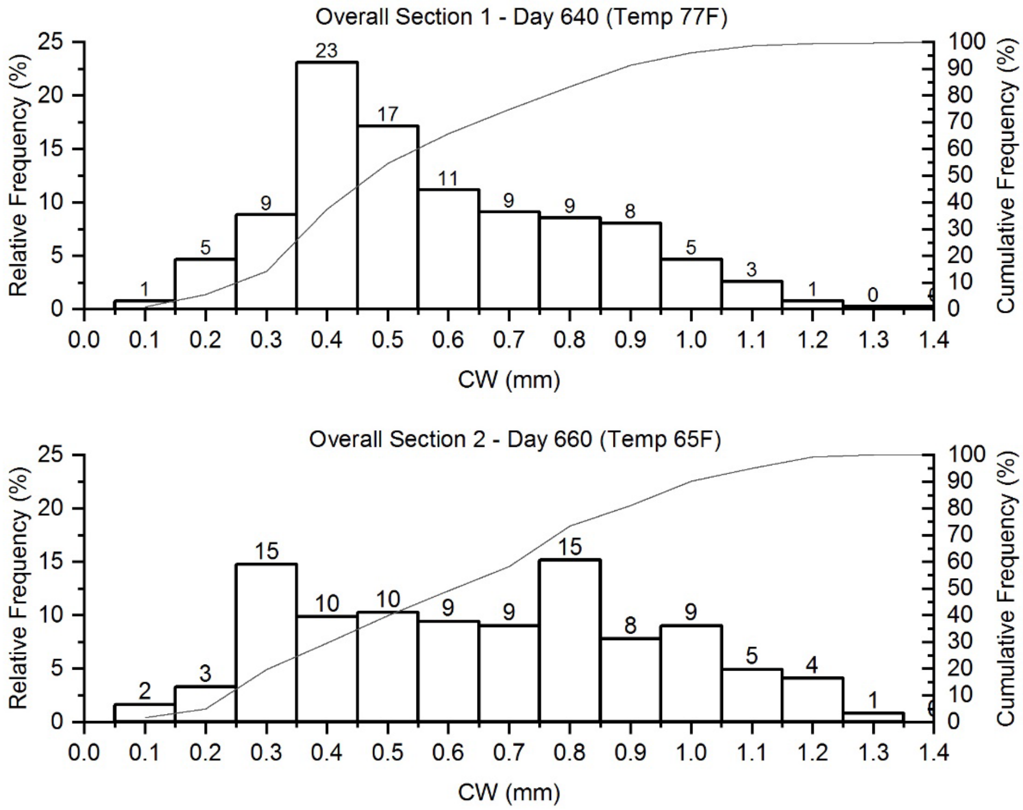


Figure 27. Tollway test sections 1 and 2 crack width distribution after more than 600 days.

Various undesirable cracks such as cluster cracks, Y-shaped cracks, and divided cracks were present in different sections of CRCP test sections as seen in Figure 28 from Tollway test sections. These types of cracks are undesirable as they may lead to localized failures in the future, such as spalling or even a punchout. From Table 19, the non-IC section 2 had almost twice the number of Y-cracks and divided cracks as well as almost three times more complex Y-cracks compared to the IC section 1. Non internally cured section 2 had more than twice number of cluster cracks (average of 5 consecutive crack spacing) per 1000 ft. compared to internally cured section 1. Reduction of the undesirable cracks in the IC section 1 showed that there was an effect of internal curing on formation of undesirable cracks. However, IC alone did not eliminate the issue of such cracking given that undesirable cracks are still present in Section 1. Section 3 consisted of only two of Y-cracks and did not have any signs of other undesirable cracks or cluster cracking. The combination of internal curing and active notching of the edge of the CRCP resulted in almost no undesirable cracks in Tollway section 3.

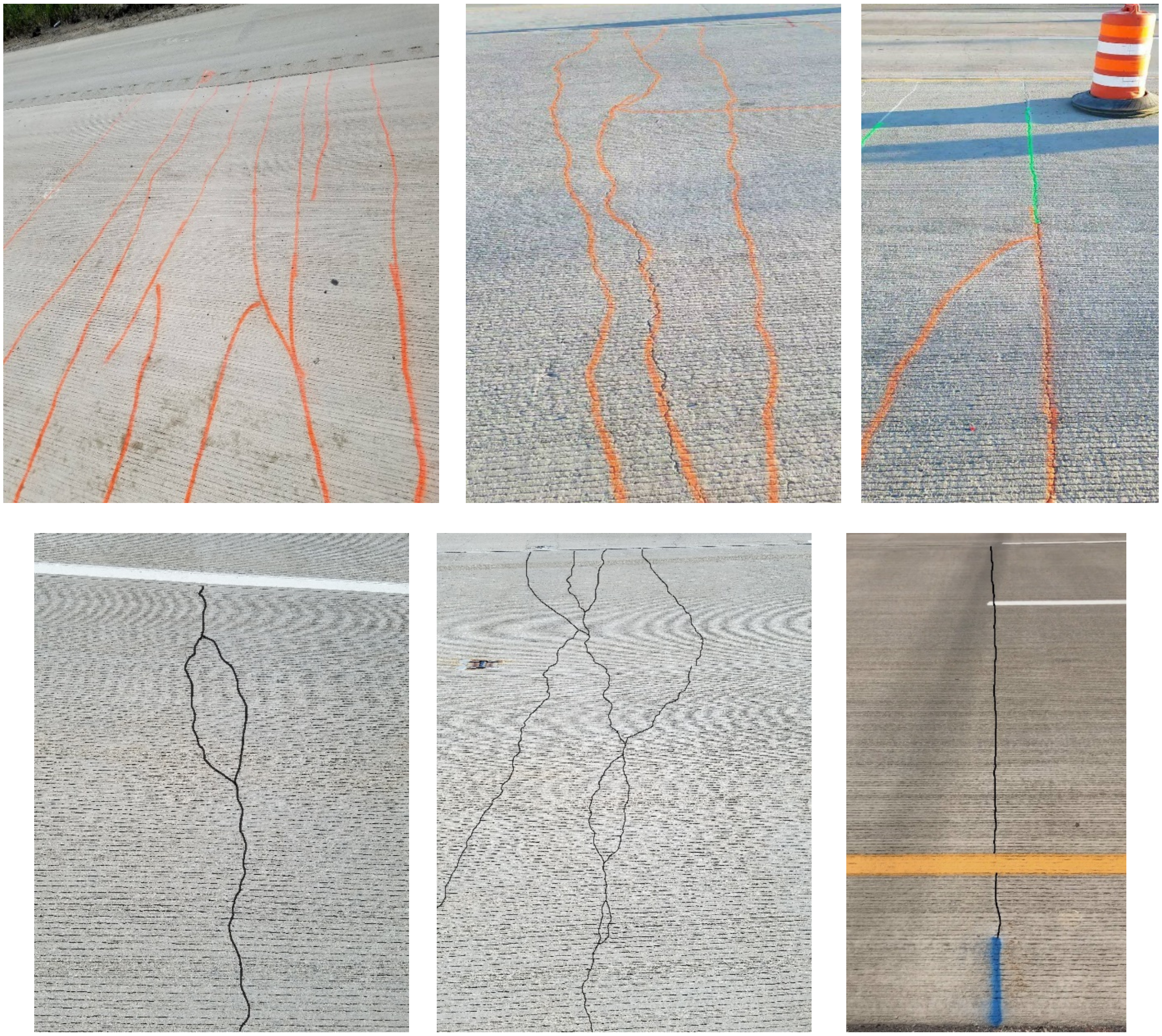


Figure 28. Complex Y-cracking with cluster cracks (top - left), cluster cracks showing three closely spaced transverse cracks (top-middle), Y-shaped crack (top-right), divided crack (bottom- left), complex y-crack (bottom-middle), straight transverse cracks seen in section 2 (bottom right).

Table 19. Undesirable and Cluster Cracks per 1000 ft

Age	Section	Y-crack	Complex-Y	Divided	“Straight”	Cluster cracks (Average of 3 cracks)	Cluster cracks (Average of 5 cracks)
640	1	13	5	5	0	10	3
660	2	22	13	10	6	19	8
375	3	2	0	0	0	0	0

5.2 SLAB THICKNESS

The slab thickness design and crack width calculations are very dependent on estimating the crack spacing. As noted previously, the crack spacing has been linked to how a CRCP performs with respect to punchouts over its life. Given the Tollway field test sections, FWD testing was performed over time to characterize the in-situ response and estimated performance of the various design features that were studied. Table 20 is a summary of the FWD tests and Illinois Tollway test section design features.

Table 20. Summary of FWD Drops for CRCP Test Section 1

Test section 1	E	D	C	B	A
Slab Thickness (in.)	10.5	10.5	10.5	10.5	10.5
Base type and thickness	2in.WMA + 4in. CTB	2in.WMA + 4in. CTB	2in.WMA + 4in. CTB	Microsurfacing + 6in. CTB	Microsurfacing + 6in. CTB
Reinforcement ratio (%)	0.80	0.80	0.58	0.58	0.58
Length of section (ft)	476	360	360	295	479
Load (lb)	9000	9000	9000	9000	9000
No. of Drops on May 2017	10	9	8	8	5
No. of Drops on August 2017	10	9	9	6	11

Figure 29 shows the backcalculated effective slab thickness for test section 1. The effective thickness represents the composite slab thickness of the CRCP slab and stabilized base layers. As seen in Figure 29, the CRCP slab of 10.5 inches is the main contributor to the effective thickness but the combined CTB layer and AC interface friction also provide several inches of equivalent slab thickness. Figure 30 shows for the monitoring period, the effective thickness did not change signficantly indicating the slab-base interface is stable. A summary of the backcalculated parameters for all the Tollway sections for all FWD tests is shown in Table 21. Load transfer across the transverse cracks in all sections is approximately 90% with the LTE versus length along the three test sections shown in Figure 31.

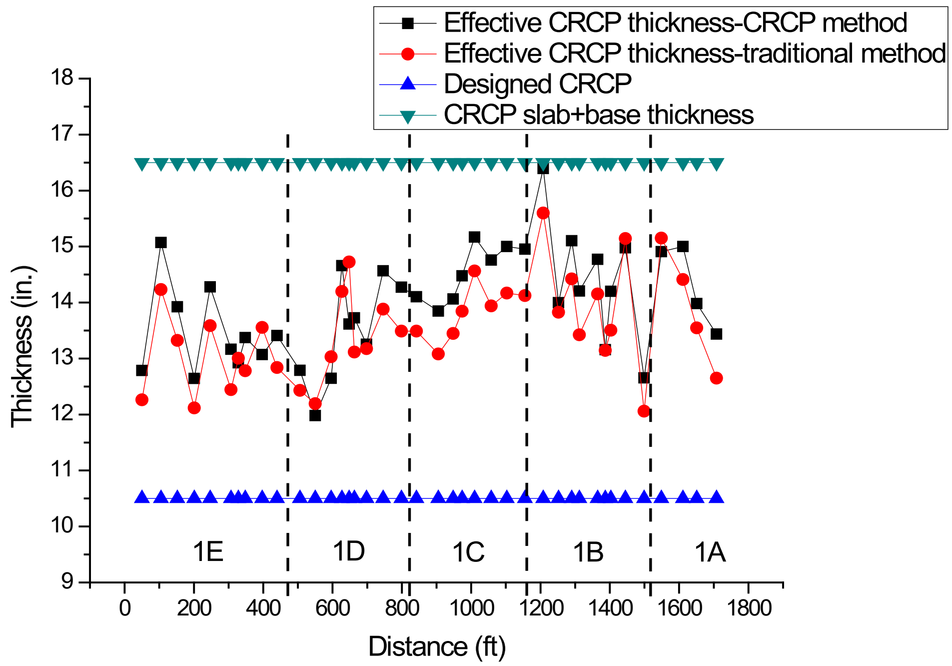
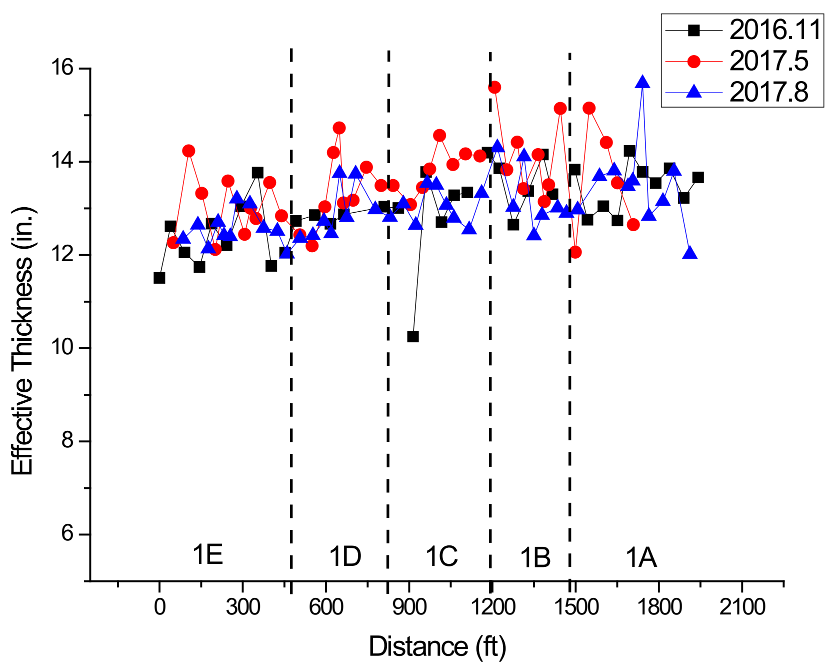
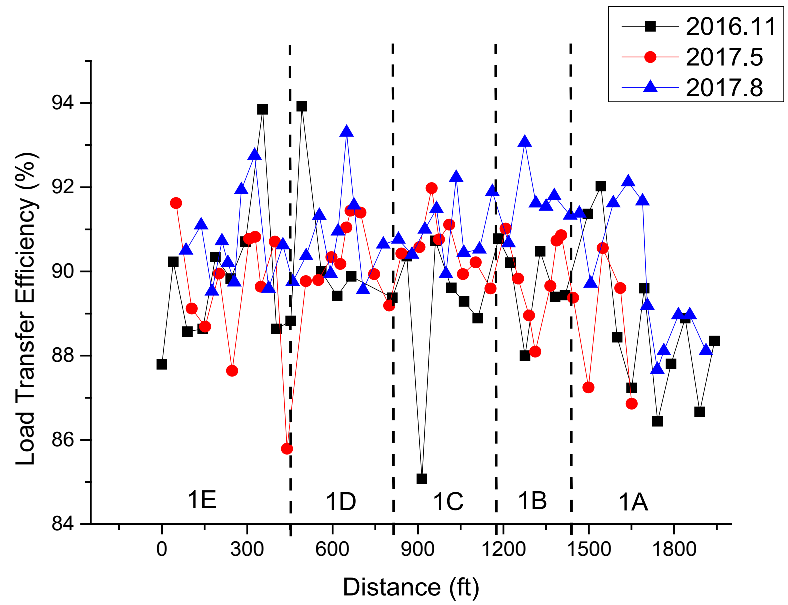


Figure 29. CRCP backcalculated effective slab thickness along the test section 1 with new backcalculation procedure for finite-sized CRCP panel sizes.

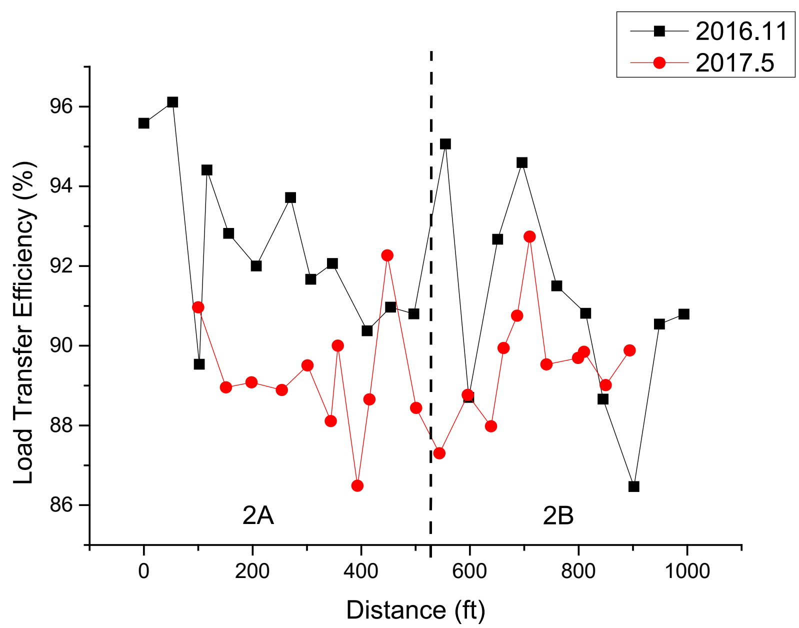


(c) Backcalculated h_{eff}

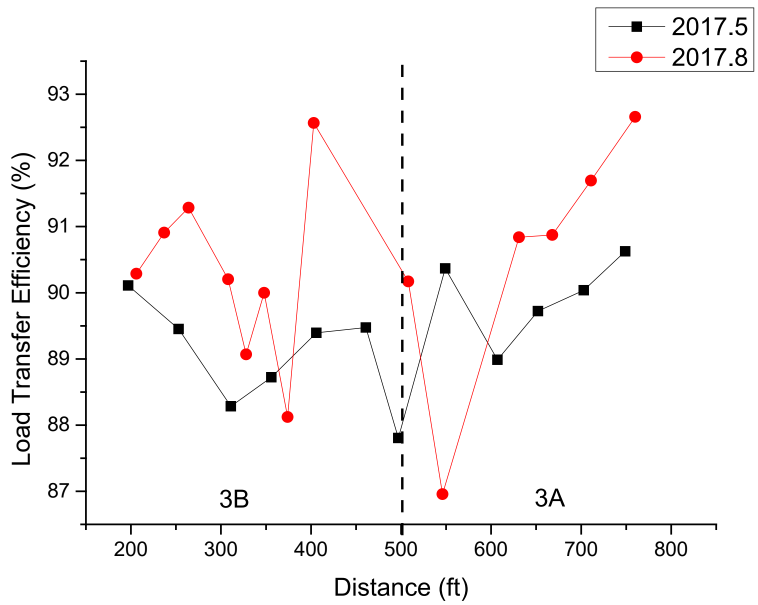
Figure 30. Backcalculated h_{eff} in section 1 over 10 month period.



(a) LTE in section 1



(b) LTE in section 2



(c) LTE in section 3

Figure 31. Load transfer efficiency in (a) Section 1, (b) Section 2, and (c) Section 3.

Table 21. FWD Summary of Backcalculated Parameters for Tollway Sections 1, 2, and 3

	***k*-value**			***h_{eff}***			***E*-value**		
	Average (psi/in.)	Standard Deviation (psi/in.)	COV(%)	Average (in.)	Standard Deviation (in.)	COV(%)	Average (10^6 psi)	Standard Deviation (psi)	COV(%)
2016.11									
1E	454.9	90.4	19.9	12.4	0.7	5.5	8.3	1.4	17.0
1D	427.1	38.1	8.9	12.7	0.3	2.4	9.0	0.6	7.0
1C	532.0	44.0	8.3	12.7	1.3	9.9	9.0	2.3	25.7
1B	519.6	33.6	6.5	13.6	0.5	4.0	10.9	1.3	11.7
1A	664.3	145.2	21.9	13.5	0.5	3.6	10.6	1.1	10.6
Overall	**527.1**	**129.1**	**24.5**	**13.0**	**0.8**	**6.4**	**9.6**	**1.7**	**18.1**
2016.11									
2A	321.9	74.8	23.2	14.1	1.0	7.3	12.2	2.7	22.3
2B	418.5	171.2	40.9	13.4	1.1	8.3	10.6	2.7	25.7
Overall	**365.8**	**136.7**	**37.4**	**13.8**	**1.1**	**8.1**	**11.5**	**2.8**	**24.8**
2017.8									
3B	378.5	52.5	13.9	10.5	0.1	1.2	7.9	0.3	3.6
3A	404.8	52.4	12.9	10.0	0.4	4.4	6.9	0.8	12.3
Overall	**393.5**	**54.0**	**13.7**	**10.2**	**0.4**	**4.1**	**7.4**	**0.8**	**11.4**

5.3 BASE THICKNESS

Generally, there are three interface bonding conditions: fully bonded, fully unbonded (slip), and partially-bonded or more appropriately, unbonded with friction. The stronger the interface friction/bond, the greater the backcalculated effective slab thickness. The new backcalculation method for CRCP applied to the Illinois Tollway test section 1 determined the average h_{eff} for sections 1A/1B with the microsurfacing/CTB was 14.3 inch and 1C to 1E with the WMA/CTB was 13.8 inches (see Table 22 for May 2017 drops). Overall, there appears to be good interface friction between the slab-base layer with the h_{eff} 3 to 4 in. over the design CRCP thickness for test section 1. The composite CRCP-base thickness is greater for the microsurfacing and 6-inch CTB than the 2-inch WMA/4-inch CTB. The interface friction for both the microsurfacing on cement treated base (CTB) and the combination of asphalt base and CTB is shown in Figure 32, with overall good interface friction existing in the first few years of the test sections.

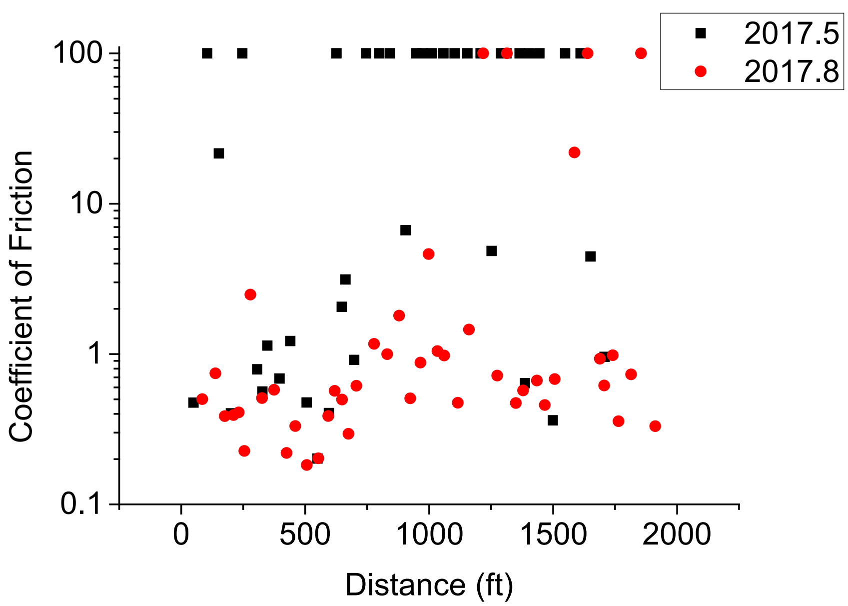


Figure 32. Non-dimensional coefficient of friction along test section 1 from both FWD tests.

Table 22. Summary of Effective Slab Thickness and Non-Dimensional Friction Coefficient for Tollway Test Section 1

Test section	FWD test on May 2017		FWD test on August 2017	
	h_{eff} (in.)	Λ*	h_{eff} (in.)	Λ*
1E	13.5	1.51	12.7	0.46
1D	13.5	1.73	12.6	0.39
1C	14.5	100	13.3	0.99
1B	14.6	100	13.5	1.15
1A	14.0	7.49	13.4	0.91

Both the interior- and edge-loaded backcalculation procedures (Zhang and Roesler 2021) were applied to a CRCP test section on the Illinois Tollway test sections. The backcalculate effective thicknesses of the CRCP with interior load testing indicated good frictional interface between the surface slab and the base layers. The testing at the edge produced results that were less realistic and needed further analysis to explain what was being measured.

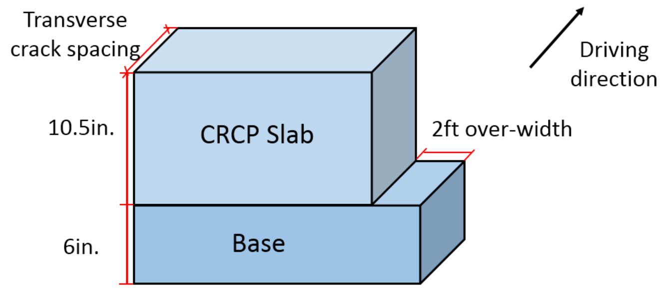


Figure 33. CRCP test section with a 2 ft extended base width.

The Tollway section had an extended base layer of 2ft, which significantly reduced the edge deflections (See Figure 33). This was interpreted in the backcalculation process as a much thicker effective thickness than reality. An improved edge backcalculation procedure to account for an extended base layer was developed using 3D finite element analysis and 2D deflection basins from Illislab. Given a 2ft extended base layer (CTB) for the range of crack spacing encoutered, the equivalent base thickness for this over-width ranged from 6.94 in. to 7.07 in., which is aproximately 1 in. higher than designed base thickness (6in.). This is one of the main reason for the higher effective thicknesses backcalculated for test section 1. Figure 34 shows the new backcalculated results given the consideration of the extra 1 in. thicker equivalent base layer.

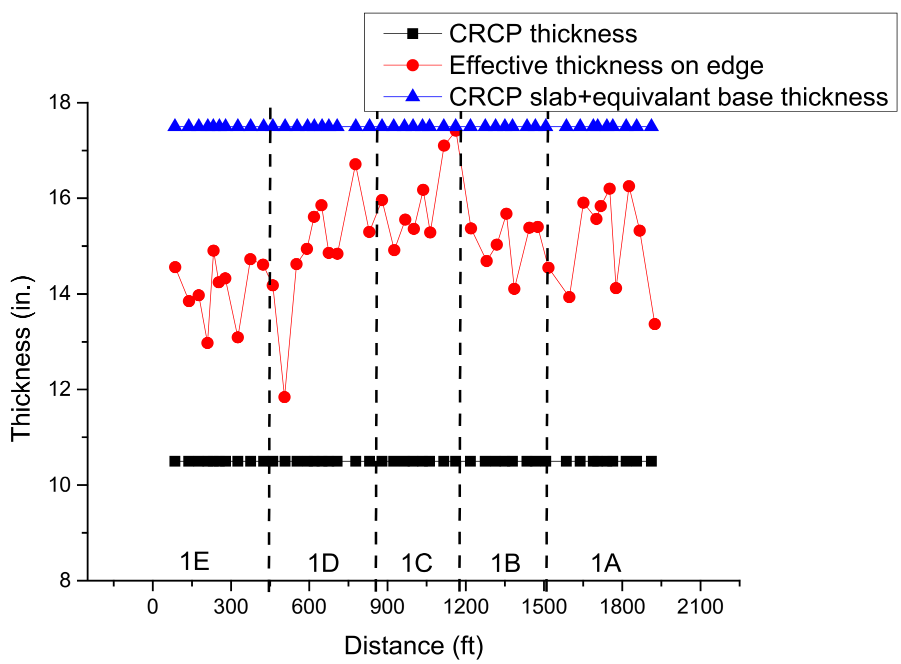


Figure 34. Backcalculated effective thickness results with the consideration of 2-ft extended base layer for test section 1.

The h_{eff} is around 4 to 5 in. thicker than the designed slab thickness as seen in Figure 34, which includes the actual CTB thickness (and WMA layer), base overwidth effect, interface friction, and consideration for finite-sized panel. The backcalculated results were within reasonable range after considering all these factors with several additional inputs to the model required to even become more accurate such as actual slab thickness, actual base thickness, and the exact spacing between full-depth transverse cracks. Overall, the edge of the CRCP shows that there is a good interface friction between the CRCP and base layer with h_{eff} 4 to 5 in. over the designed CRCP thickness at the edge for this project.

5.4 STEEL CONTENT AND BAR SIZE

Over the years multiple studies in the US and internationally has shown that steel content affects the performance of CRCP. Steel contents that become too low have lead to premature punchout formation through wider cracks and loss in crack load transfer efficiency. The field test sections had two steel contents with the idea of finding an optimal steel content that provided the appropriate crack widths without excessive amounts of steel. A steel content of 0.80% is generally thought to be an upper limit and 0.6% steel content a lower limit. The ATREL continuously reinforced concrete beam (CRCB) section and the Illinois Tollway field test sections on Illinois Rte 390 both demonstrated that IC statistically lowers crack width relative to traditional concrete mixtures. Given the original hypothesis that IC mechanism and/or macro-fibers will tighten the crack widths, the maximum steel content can likely be lower than the upper limit without sacrificing CRCP performance.

Minimizing the bar size at a constant steel content has shown to lower crack spacing and width. Therefore, a design based on minimizing the bar size for the selected steel content will increase performance of the CRCP by engaging a larger volume of the concrete into restraint and intersecting cracks with closer spaced rebars.

5.5 TRANSVERSE POST-TENSIONING OF STEEL SUPPORT

Speakmon et al. (2021) in Volume 5 of the report summarize the design and cost-effectiveness of employing a transverse post-tensioning system to CRCP to reduce the intitial costs by applying a transverse compression into the CRCP slab system. In evaluation of this rigid bar post-tensioning design, the case studies performed have shown promising results in effectively distributing the stress and transferring the stress into the CRCP. The initial research implemented the idea on a standard pavement design without altering the structural design in any major way. The initial goals were to test the effectiveness of the system itself, along with researching the distribution of stresses through the bar and across the pavement. Currently, the pavements are being tested to determine how much stress can actually be implemented with each method or option. The effectiveness of the system was evaluated by using a number of different strain gauges placed on the internal steel within the CRCP.

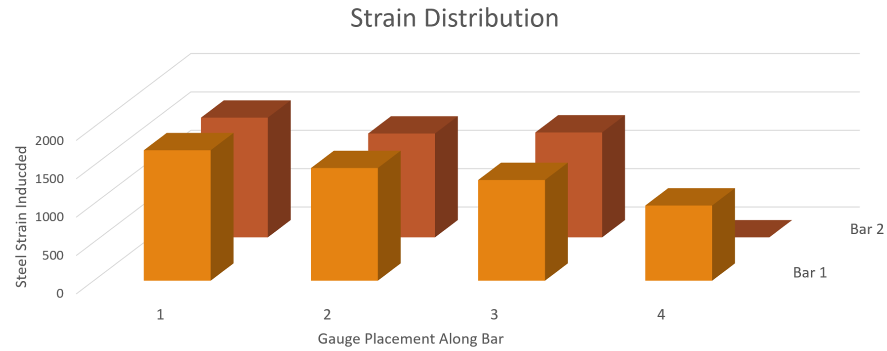


Figure 35. Victoria TX post-tensioning strain distribution across bar.

Figure 35 shows the initial Victoria, TX testing results from tensioning the bars for the first time with strain distributed for them most part successfully across the bar. During he second visit to tension the Victoria section, a greater understanding for the relaxation and strain distribution was found. The solution for this was putting a sleeve over the bar to reduce the steel-concrete bonding issue. After tensioning the bar a second time around, the strain readings were much more precise showing the stress distributing throughout the entire bar and therefore throughout the pavement.

The data collected from the TTMU/TTI Rellis Campus Test slab showed a few conclusions. The stress distributed to both ends of the bar was expected as well as a higher stress induced into the concrete closer to the ends. The decrease in stress at the middle of the slab was more dramatic than expected but there was still some stress induction in the middle of the slab. Strain gauges implemented at the top and bottom of the CRCP showed changes in strain representative of curling but did not show any differential. This can be interpreted as reduced or minimal curling of the slab with the post-tensioning showing promise in negating transverse curling. The post-tensioning findings also show potential in the reduction of longitudinal cracking and to hold slabs tightly together, increasing load transfer, and minimizing slab movement. The findings of the test sections also showed that each post-tensioning system is feasible and a field test section should be tried on a higher volume roadway.

CHAPTER 6: CONSTRUCTION METHODS

6.1 ACTIVE CURING MANAGEMENT

Curing is an important step in developing long-lasting concrete since it promotes hydration of the cement leading to strength and durability. A number of specifications are available to curing, and a vareity of test methods have been used to assess their efficiency. In this study, neutron radiography was used to study curing efficiency with FLWA and SAP, using different curing techniques (Montanari et al. 2021). Neutron radiography is an experimental technique that can be used to measure the water contained in hydrated cements, determine the degree of hydration in a cementitious system, and determine the 'curing affected zone' (CAZ) in a specimen. Internal curing was demonstrated to improve the curing effectiveness and degree of hydration near the surface of the concrete where external drying is taking place, high stresses exist, and where durability performance is demanded.

6.2 ACTIVE CRACK CONTROL

Active crack control has seen success on several projects over the past 20 years but has never been seriously implemented by a U.S. transportation agency (Zollinger et al. 1998; McCullough et al 1999; Kohler and Roesler 2004; Ren et al. 2014; Ren 2015). The early work in the U.S. on active crack control has recently been adopted in Belgium CRCP with a modification to minimize the depth and length of the sawcut. Ren (2015) and Ren et al. (2014) studied partial width and depth notching at CRCP edges as a means of cost effectively implementing the idea and minimizing undesirable cracks.

Table 23 is the success at the Univeristy of Illinois ATREL facility with the CRCB and active cracking. For these sections on the end 60 ft were notched (terminal joint notching) to minimize movements at the terminal end and improve the crack spacing development. At 300 days after construction 100% of the notches cut had active cracks propagating from them for different concrete materials (IC and non-IC) and steel contents (0.56% and 0.76%).

Table 23. Cracks Propagating through Sawcut Notches in CRCB at UIUC ATREL

DAYS	6	14	49	79	182	300	570	917
#6 - Section 2A	0	0	4	4	4	10	10	10
#6 - Section 2D	3	5	9	9	9	10	10	10
#6 – Section 2 Total	*3*	*5*	*13*	*13*	*13*	*20*	*20*	*20*
#7 - Section 1A	0	0	0	0	10	10	10	10
#7 - Section 1D	5	8	9	9	9	10	10	10
#7 – Section 1 Total	5	8	9	9	19	20	20	20

The partial width and depth notches were fully implemented on the Illinois Tollway field section 3 (2ft long and 2 inches wide from the CRCP edge) is shown in Figure 9 in Chapter 2. Partial-width saw cut notches were also completed near the terminal joints at the ends of Tollway section 1 (1A and 1E). Tables 24 and 25 show the total number of cracks passing through sawcuts in section 1 and section 3 at different ages. There are total 10 sawcuts each in section 1A and 1E (See Table 24). The first sawcut activation was observed in 1E around day 119. It took around 300 days for the first sawcut to activate in section 1A. At day 640, only 30% of sawcuts in Section 1A and 40% in section 1E were activated. Several plausible reasons for cracks not propagating from these notches are late sawcut timing, insufficient notch depth and width, and insufficient slab-base friction to generate tensile stress in the concrete to propagate a crack vertically.

Table 24. Total Number of Cracks Passing through Sawcuts near the Ends of Section 1

	Days						
Section	**19**	**80**	**119**	**259**	**300**	**420**	**640**
1A (10 cuts)	0	0	0	0	1 (10%)	2 (20%)	3 (30%)
1E (10 cuts)	0	0	1 (10%)	1 (10%)	1 (10%)	1 (10%)	4 (40%)

Table 25 clearly demonstrates the success of the partial-width and depth notching on establishing cracks early and more regularly. After one year, over 70% of the cracks have propagated at the notches in section 3. Only a few transverse cracks in section 3 have propagated from location without notch. Figures 36 and 37 demonstrate the small crack and narrow distribution of widths when cracks activate early and at regular intervals. As see in Figure 22 (Chapter 5) after a year, Tollway section 3 had only 8% of crack spacing lower than 3 ft. indicating effectiveness of active cracking in reducing undesirable crack patterns. Additionally in section 3, the average crack spacing was around multiples of four feet which matched the distance between partial width notches.

Table 25. Total Number of Cracks Passing through Sawcuts in Section 3

	Days			
Section	**22**	**65**	**157**	**375**
3A (146 cuts)	10 (7%)	35 (24%)	60 (41%)	101 (69%)
3B (99 cuts)	12 (12%)	21 (21%)	36 (37%)	73 (74%)

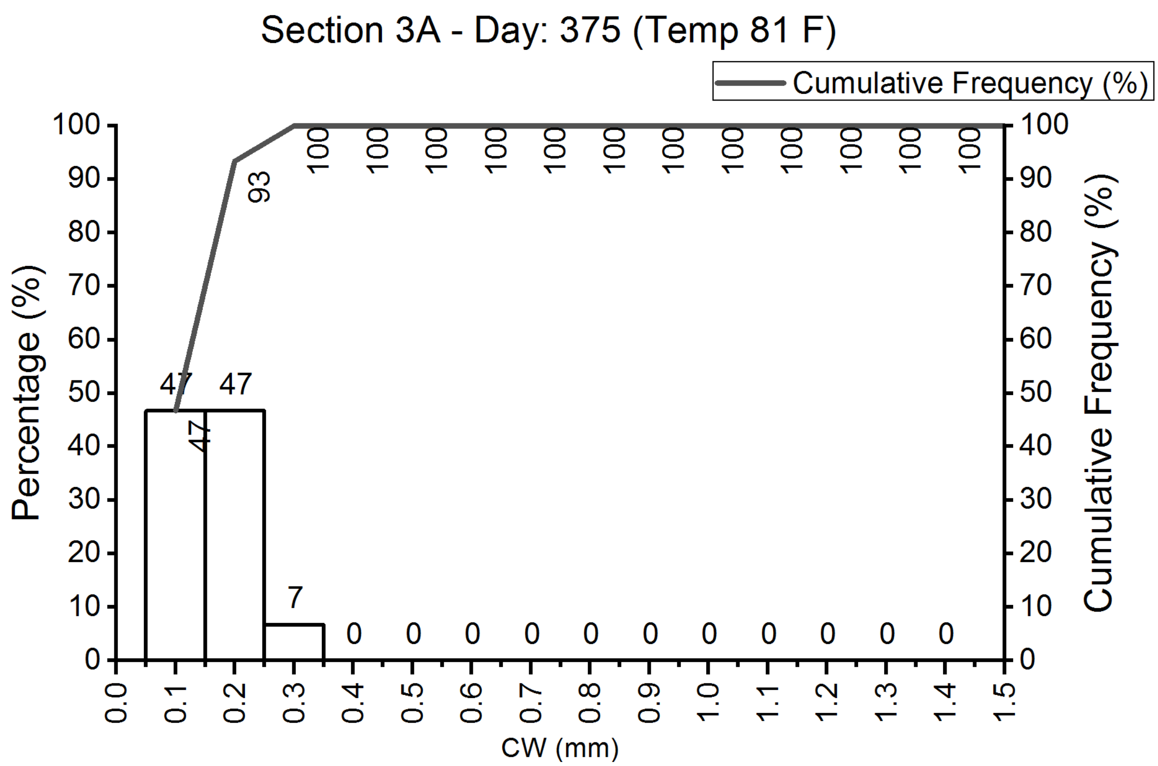


Figure 36. Crack width distribution for Section 3A on day 375.

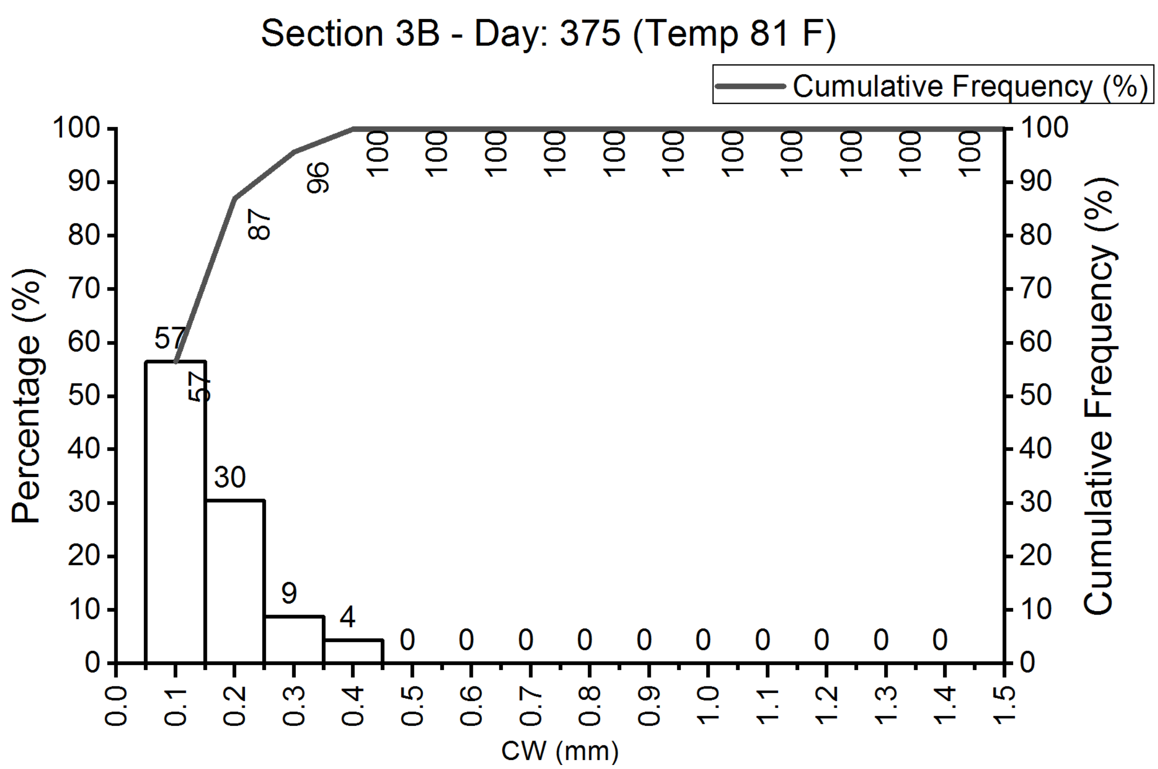


Figure 37. Crack width distribution for Section 3B on day 375.

6.3 RAPID STEEL PLACEMENT

Presently, the technologies to place steel whether through an improved tube placement method or rolling out cages of pre-fabricated steel were not assessed to produce the efficiency and cost savings. Implementation on a Tollway project at this time and subsequently was abandoned from further analyses.

6.4 TERMINAL END JOINT SYSTEMS

The performance of the terminal joint movement in section 1 (see Tables 24 and 25) and the improved Illinois Tollway terminal joint design is described in volume 5 of Speakmon et al. (2021). The new terminal joint design is much simpler to construct and doesn’t involve constructing lugs or installing a wide flange beam to accommodate the CRCP movement. The terminal end joint design also takes adavantage of surface notching near the end of the CRCP to distribute movement out to avoid excessive movements directly at the end of the terminal joint.

CHAPTER 7: SUMMARY OF RESEARCH FINDINGS

Based on the research from the four volumes (Dahal et al. 2021; Zhang and Roesler 2021; Montanari et al. 2021, and Speakmon et al. 2021), Table 26 summarizes the key finding for a list of design, material and construction features evaluated during the study. Table 26 also provides limitations to the finding to avoid over generalizing a finding. Table 27 lists the resultant crack properties that are an outcome of design, material, and construction decisions coupled with the environmental conditions.

Table 26. Summary of CRCP Finding for a Specific Design, Material, or Construction Feature Controlled by the Engineer

Parameter	Finding	Limitation
Internal Curing	• Decreases crack width on average 25%. • Reduction in number of undesirable cracks. • Improvement in near surface curing effectiveness. • Educate contractor on properly managing moisture on FLWA stockpiles. • New method to calculate required FLWA to provide internal curing. • SAP introduction show promise for IC.	• Does not eliminate all undesirable cracking. • Do not expect to see significant reduction in slab thickness with FLWA addition. • Need additional field studies to implement SAP in concrete paving mixtures and verify field control.
Active Cracking	• Significantly controls undesirable cracks. • Creates straighter and tighter cracks at regular spacing. • Early crack activation when done throughout the length.	• Determining proper timing and depth of surface notching is important to activation. • Optimal distance between the notches needs to be researched.
Macro-Fibers	• Tighter crack widths. • Provides tighter crack widths in lower steel content and comparable to crack widths of higher steel content without fibers.	• Adds additional cost but with a potential reduction in steel.
Steel Content	• Smaller bar size (such as #6) preferred for same percentage of rebar to restrain more concrete volume. • Higher steel content (preferably more than 0.70%) provides tighter cracking. • Most significant factor in crack spacing and width development. • Black bar not found to affect long-term performance both in terms of crack properties and corrosion durability.	• Expensive to incorporate too much steel but lowering steel too much can create larger crack width that can potentially reduce the age of pavement.

Parameter	Finding	Limitation
Concrete Mixtures	• Performance-based specification needed to proportion paving mixtures for internal curing, lower cement content, and increased SCM content.	• Need to maintain freeze-thaw resistance for durability of CRCP.
Construction time / temperature	• Temperature during concrete paving is very important to crack development and crack width magnitudes. • Lower temperature during construction can significantly reduce the crack width as seen in Section 3.	• Quality of concrete paving must be monitored closely especially at night. • Other environmental factor such as wind speed and relative humidity affect performance as well.
Slab and Base Thickness	• Slab-Base composite action provides several inches of equivalent CRCP slab thickness. • Higher interface friction results in greater effective thickness. • Base erodibility testing with Hamburg Wheel Track test is effective in evaluating potential stabilized base layers.	• Need to maintain proper long term friction. • Base with lower erodibility needed for maintaining higher effective CRCP thickness.
Terminal joint	• New terminal joint design simpler to construct. • Active cracking at end of terminal joint can reduce end movements .	• Tollway test sections were not as long as typical field sections. • Deeper notches and as early as possible required to propagate a vertical crack.
Post-Tensioning	• Preliminary results demonstrate acceptable transverse strain distribution in the steel and concrete stress while tensioning the transverse bar. • Initial results demonstrate a reduction in transverse curling.	• Need more field test sections to demonstrate CRCP post-tensioning technical and cost effectiveness.

Table 27. Summary of CRCP Finding for a Specific Features as a Result of the Design, Materials, and Construction Specified

Parameter	Finding	Limitations
Crack Width	• Reduced by: ○ Higher steel content ○ Internal Curing with FLWA ○ Lower steel content with macro-fibers ○ Cooler construction temperature, e.g., Section 3	• May varying depending on steel-concrete interaction with epoxy-coated steel. • May vary with a different base type.
Crack Spacing	• Initially no statistical difference between IC and non-IC test sections. • Active crack control provides uniform and straighter cracks. • Internal curing reduced cluster cracking.	• IC alone does not eliminate cluster formation. • Excessive surface notching spacing may lead to undesirable cracks.
Undesirable Cracks	• Internal curing significantly reduces such transverse cracks. • Active cracking with internal curing almost eliminates such cracks.	• Crack initiation and straight propagation needed – which can be achieved by active cracking.

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