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Concrete Pavement Joint Deterioration

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16. Abstract

Concrete pavements are an important part of our national infrastructure. In recent years the relatively small number of reported joints deteriorating prematurely in concrete pavements around Indiana has increased. Changes over the past 45 years in INDOT specification, pavement materials, designs and construction practices, and current de-icing materials were examined and related to the durability of concrete at the joints of existing pavements. A survey of concrete pavements across the state revealed that no pavements from the two southern districts less than 40 years old showed this distress except in more recently placed patches.

Cores were retrieved from the joints and mid-panel of 11 pavement sections that represented different materials, ages, construction, deicer exposure, and different levels of deterioration, from non-deteriorated concrete to concrete with severe deterioration at the joints. The pavement base drained well at the mid-panel of most pavements but was reduced at the joints for over half the pavements with the most severe joint deterioration associated with the slowest drainage. None of the concrete had an air void system that met all the criteria recommended for FT durable concrete but was better at the mid-panel than at the joints. Infilling and lining of the entrained air voids with ettringite and some Friedel's salt was more common near the joints and could account for the reduced air void system. The FT testing did not correlate directly with the air void parameters but generally mid-panel samples did test as more durable than joints. Evidence from the presence of unhydrated cement grains suggested that the concrete at the joint face was not fully cured. One pavement section that did not have fly ash had worse deterioration than the panels nearby that had fly ash and calcium hydrate was more noticeable in the concrete from joints with severe deterioration.

Several variables were identified that influence the durability of the concrete at the joints and there may be other variables that were beyond the scope or capacity of this study. In summary this study identified the following variables likely influenced the durability of the concrete at the joints: the drainability of the base at the joints, original air void system, reduced air void parameters due to lining and infilling of the air voids with secondary minerals, poor hydration of the concrete at the joint face and increased moisture at the joint.

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EXECUTIVE SUMMARY

CONCRETE PAVEMENT JOINT DETERIORATION

Introduction

Concrete pavements are an important part of our national infrastructure. In recent years the number of reported joints deteriorating prematurely in concrete pavements around Indiana has increased. Changes over the past 45 years in INDOT specifications, pavement materials and design, construction practices, and deicing materials were examined and related to the durability of concrete at the joints of existing pavements.

Cores were retrieved and examined from the joints and mid-panel of 11 pavement sections that represented different materials, ages, construction, deicer exposure, and levels of deterioration, from non-deteriorated concrete to concrete with severe deterioration at the joints.

Findings

Several variables were identified that influence the durability of concrete at the joints: use of fly ash, joint sealer type, saw cut configurations, water-to-cementitious ratio (w/cm), 7-day flexural strength acceptance criteria, minimum cement content, tie bar spacing and size, target percentage air, and minimum percentage air before failure.

The physical properties and chemistry of cements have changed over the years. The fineness has increased for INDOT cements as well as cements used across the country.

- The amount of C_3S has increased, while the amount of C_2S has decreased.
- Since 1954, the 1-day through 28-day strengths have all increased; the 1-day and 3-day strengths have increased the most dramatically, resulting in increased early-age rate of strength gains.
- The 7-day strength values have been the most consistent across all cements examined since 1990.
- The increase in both fineness and C_3S have contributed to the dramatic increase in 1-day and 3-day strengths but also can contribute to higher amounts of CH in the concrete.
- The sulfates also have increased to counteract set problems that can occur with finer cements and more readily available aluminates.

A survey of concrete pavements across Indiana revealed that no pavements less than 40 years old from the two southern districts showed this distress, except in more recently placed patches. These districts not only experience a less harsh freeze-thaw (FT) environment but also use lower amounts of deicers. Other evidence from field and laboratory analysis of existing concrete pavements includes the following:

- The pavement base drained well at the mid-panel of most pavements but was reduced at the joints for over half the pavements, with the most severe joint deterioration associated with the slowest drainage.
- None of the concrete had a measured air void system that met all the criteria recommended for FT durable concrete, but the air void systems were better at the mid-panel than at the joints. Infilling and lining of the entrained air voids with ettringite and some Friedel's salt was more common near the joints and could account for the reduced air void system. The FT testing did not correlate directly with the air void parameters, but generally mid-panel samples did test as more durable than joints.
- The presence of unhydrated cement grains suggested that the concrete at the joint face was not always fully cured.
- One pavement section that did not have fly ash had worse deterioration than the panels nearby that had fly ash.
- Calcium hydroxide was more noticeable in the concrete from joints with severe deterioration.

In summary, this study identified that one or more of the following variables likely influenced the durability of the concrete at the joints examined: the drainability of the base at the joints, original air void system, reduced air void parameters due to lining and infilling of the air voids with secondary minerals, compromised hydration of the concrete at the joint face, increased moisture at the joint.

Implementation

Steps to consider that could reduce the potential for concrete to deteriorate at the joint include the following:

- *Fly ash* or other SCM that provides additional silica as part of the cementitious mixture can help convert CH into CSH, which is especially critical if the cement has a higher C_3S:C_2S ratio—common in modern cements. Many modern cements are more susceptible to higher heats of hydration. SCMs that reduce the heat of hydration are especially valuable when concrete is placed during high ambient temperatures.
- *Sealing* of joints without a backer rod may reduce the amount of moisture held at the joint face that contributes to the concrete reaching the critical saturation level that renders it susceptible to FT damage. Treating the joint face with a silane or other penetrating waterproof sealer soon after sawing may improve the curing of the concrete at the joint face, making the concrete more durable, and may reduce the potential of the concrete to become critically saturated throughout the life of the joint.
- The *air void system* is commonly reduced in older pavements. Adopting practices that give an excellent original air void system is valuable, encouraging spacing factors and specific surface that are much better than marginal. These parameters are critical to long-term durability of concrete exposed to a harsh FT environment and deicers. A balance between optimal air for long-term durability and meeting early strengths requirements needs to be considered.
- Ensuring the hydraulic conductivity of the base is adequate, especially at the joints, and remains good throughout the life of the pavement.

In existing pavements, steps that reduce the amount of moisture at the joint will likely reduce the potential for or rate of concrete deterioration at the joint. Practices recommended by other researchers that are applicable include removing the backer rod and sealing the joint face with a silane, siloxane, or other penetrating waterproofing sealer.

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1. INTRODUCTION

1.1 Background

Concrete pavements are an important part of our national transportation system providing long-term high quality reliable structures. However, in recent years the number of reported joints deteriorating prematurely in concrete pavements around Indiana has increased. The deterioration manifests itself as spalling, crumbling and erosion of concrete in the joint area, primarily along the longitudinal saw cut joints. Previous INDOT sponsored research projects attributed damage to excessive moisture at the damaged joints, in-filling of the air void system with secondary hydration products and higher rates of absorption all of which lead to critical saturation and low freeze-thaw durability. These issues have been described in the previous JTRP report (Arribas-Colón, Radliński, Olek, & Whiting, 2012). In addition, factors such as poor drainage of the sub-base, enhanced saturation in the presence of deicing chemicals, abundance of unhydrated cement grains within 5 mm of the sawn joint face (suggesting poor curing at the joint face), failure to develop or to sufficiently open the crack under the saw cut and entrapment of water under partially damaged sealer in the joint also were observed but not thoroughly evaluated as to if or how they may have contributed to the accelerated damage (Arribas-Colón et al., 2012). In summary, the previous study made it clear that the observed damage of the joints is a result of several combined factors but it did not yield information to allow for a comprehensive explanation of all processes leading to the observed deterioration.

The Portland Cement Association (PCA) promotes recommendations for satisfactory air void system parameters needed for adequate freeze-thaw (FT) durability, citing work by Powers from 1954 and 1965 (Kosmatka, Kerkhoff, & Panarese, 2002). Studies completed in more recent years by other researchers and agencies that may relate to concrete deterioration similar to that examined in this study suggests that deicers may play a role (Sutter et al., 2008) and the concrete deterioration at the joints may be related to freezing and thawing and an inadequate or compromised air void system. The 2006 FHWA Report HRT-06-117, *Freeze-Thaw Resistance of Concrete With Marginal Air Content* (Tanesi & Meineinger, 2006) suggested that consistent and good freeze-thaw (FT) resistance may be achieved with the air entraining admixture (AEA) vinsol resin at 3.5% air content and higher. However the one synthetic air entraining agent (AEA) tested did not produce similarly good results at similar total air contents.

Kanga, Hansena, and Borgnakke (2012) showed field concrete with less than 6% total air but had other air void parameters consider acceptable (spacing factors <200 microns [<0.008 in.] and specific surface >24.5 mm^{-1} [>600 in^{-1}]) did not exhibit concrete deterioration at the joint. As shown in this study and previous projects (Arribas-Colón et al., 2012) very few pavements examined had existing air void systems that met all the criteria PCA recommends for FT durability. Perhaps the air void system parameters that are adequate for FT durability need to be reviewed considering the changes and advances in mix designs, cementitious materials, air entraining admixtures and deicers.

1.2 Research Objectives

Several factors may contribute to the increased occurrence of early joint deterioration. The focus of this *Concrete Pavement Joint Deterioration* study was to examine the changes that have taken place over the past 45 years in the type and properties of pavement materials, in the design and construction practices, and in the type and usage of deicers, and then link these changes with conditions of existing pavements with a focus on the presence or absence of deterioration at the joint. The ultimate goal was to identify the influence of local materials, practices and specifications on the durability of joints.

1.3 Scope of Work

This study identified several changes in concrete pavement materials, designs, construction practices, deicing practices and INDOT specifications and established a timeline for many of these changes. Pavement sections were identified that reflect the use of different practices and materials, field observations were made, cores retrieved from selected pavements, and concrete examined both megascopically and microscopically. Samples cut from several cores were subjected to freeze-thaw cycles, tested for permeability and the air void systems analyzed. Test results, observations and collected information were analyzed and compared to the condition of the concrete, both at the joint and mid-panel.

In addition, a laboratory study was initiated to examine the influence of changing temperature, saturation level and air content of hardened concrete on the bulk conductivity of concrete specimens in order to evaluate the potential utility of the bulk conductivity test for identifying properties of existing concrete. Electrical testing is relatively easy to perform and quick to complete. Therefore, a laboratory study was initiated with a goal to develop a method to differentiate between deteriorated and well-performing joint cores using electrical resistivity/conductivity measurements. The influence of changing temperature, saturation level, and air content of hardened concrete on bulk conductivity of concrete was studied. However, additional work is needed to correlate the conductivity of concrete with the potential for it experiencing deterioration. SPR-3623 (*Variability Analysis of the Bulk Resistivity Measured Using Concrete Cylinders*) and SPR-3509 (*Early Detection of Joint Distress in Portland Cement Concrete Pavements*) have additional information on the electrical testing of concrete which could help develop standardized testing procedure in the future.

2. HISTORICAL PERSPECTIVE

To better understand how concrete pavement joint performance related to differences in materials, design, environment and construction practices specific construction records and materials information are needed. Unfortunately these records and information were not available for INDOT pavements. Since the year built usually was known some assumptions could be made based on the prevailing INDOT specifications, and materials and practices common at the time of construction. Therefore the changes in INDOT specifications with time were thoroughly reviewed, as were changes in cement chemistry and deicers used. Examining changes in other materials such as fly ash and admixtures were beyond the scope of this study.

2.1 INDOT Specifications and Construction Practices

Changes in construction practices were not documented but many changes in practices were reflected in changes in INDOT's specifications. The INDOT Stan dard Specifications books from 1952 through 2012 were reviewed, and specification changes related to concrete pavement mixtures and joints were documented by industry Study Advisory Committee (SAC) members Mike Byers (American Concrete Pavement Association, ACPA) and Chris Tull (consultant; president of CRT Consulting). Table 2.1 summarizes these findings.

Some of the significant changes that may relate to joint durability include:

- **1985:** First mention of the use of fly ash (FA) as supplementary cementitious material
- **1988:** First mention of silicone joint sealer being used
- **1995:** Double saw cut at the transverse contraction joints (T-jt) introduced
- **1995:** QA option established for PCCP mixtures and construction which included:
  - Reducing the water-to-cementitious ratio (w/cm) to ≤ 0.45,
  - Setting the acceptance criteria of 7-day flexural strength $\geq$ 570 psi
  - Reducing the minimum cement content to 440 lbs/yd^3
- **1999:** Single saw cut for longitudinal joints (L-jt), tie bar spacing of 3′ and the tie bar size adjusted for pavement thickness were established
- **2006:** Target % air increased to 5.7%–8.9% and the depth of saw cut for both T-jt and L-jt sawn joints became T/3 (1/3 of the pavement thickness). (Note: L-jt depth of saw cut changed to T/3 in 2005. For a period of time the saw cut for the L-jt was cut deeper than the saw cut of the T-jt (T/3 vs. T/4 respectively) encouraging water to drain from the T-jt into the L-jt.)
- **2012:** Minimum % air before failure raised; tie bar size decreased for thicker slabs

2.2 Changes in Cement

The chemistry, fineness and strength development of cements have changed over the years in response to both changes in technology and demands for earlier higher strengths. Users and contractors could strip forms earlier and get on slabs sooner with earlier higher strengths thereby increasing productivity. Three publications provided average cement properties from over a hundred plants across the US from years 1953–54 (Clifton & Mathey, 1971), 1994 (Gebhardt, 1995) and 2004 (Tennis & Bhatty, 2005). The published data for Type I cements were compared to data from the Mill Certificates on file at INDOT for two Type 1 cements commonly used for concrete pavements in Indiana from 1990–2010. INDOT Cement A refers to the Lehigh Mitchell plant and cement B refers to the Lonestar/Buzzi Greencastle plant.

2.2.1 Fineness

Generally, for a given chemistry, the finer the cement the more quickly it reacts and the higher the rate of strength gain (Neville, 1997). Both the national studies and the INDOT data show that the average fineness of cement, measured as Blaine (cm^2/g), has increased over the years (as shown in Figure 2.1). However, the range and variability of fineness values has decreased since 1954 (as shown in Table 2.2). Over the years examined, cement A had a higher Blaine than cement B, and the fineness of cement A was more similar to the national averages. Overall both INDOT cements fell within the range of values measured nationwide.

2.2.2 Strength

Reported strength test data for cements are based on compressive strength tests of 2″ mortar cubes performed in accordance with ASTM C 109. As shown in Figure 2.2, the average compressive strengths for ages 1-day through 28-day have increased for cements produced in 1994 and 2004 compared to those produced in 1953–54 (Clifton & Mathey, 1971; Gebhardt, 1995; Tennis & Bhatty, 2005).

The rate of the strength gain has changed over the years as shown in the slope of the lines in Figure 2.2 and more specifically as change in psi per day in Figure 2.3. The early strength gain between day 1 and day 3 shows the most dramatic change with a significant increase over the years. Whereas the rate of strength change between 3 and 7 days, and between 7 and 28 days has actually decreased over the years. Therefore the higher 28-day strengths seen in more recent years can be attributed to the higher rate of strength gained in the first 3 days.

The strengths reported for INDOT cements A and B from 1990 to 2010 also have increased compared to the national averages from 1953–54 (as shown in Figure 2.4). Comparing INDOT cement A with B, the strength results for A are more consistent throughout the years examined. The slight decrease in the 1-day strength with time for cement A while 3-day strengths remain steady suggests that the rate of strength gain from 1 to 3 days has increased. Similar to the trend in fineness, cement A strength data also appears more similar to the national data than cement B data.

TABLE 2.1
Summary of changes in INDOT concrete pavement specifications.

Year	w/cm	Air	Cure	Cement (lbs/yd^3 min)	Flexural Strength (psi)	Materials	Admixtures	Joints (Jt)	Other
1952	≤0.487	3%–6%	Wet 96 hrs	564				Hand tooled, asphalt ribbon	JRCP
1957	≤0.487	4%–7%	Same	564			AEA vinsol resin	Hand tooled, asphalt ribbon	JRCP
1963	≤0.487	Same	Wht pigmt CC mentioned	564	550 psi opening	#2 & #5 Agg; -2.5″ top sz		Sawn joints mentioned	JRCP; mix time = 50s
1969	≤0.487	5%–9%		564	550 psi opening		WR allowed	Joint filler 905 Introduced	JPCP and JRCP
1974	≤0.487	5%–8%		564	550 psi opening	#5 & #8 Agg; -1.5″ top sz		905 Joint filler removed	JPCP and CRCP
1985	≤0.487	Same		564	550 psi opening	FA[a]		No silicone mentioned	JPCP and CRCP: no mix time mentioned
1988	≤0.487	Same		564	550 psi opening	F-FA 15%[a] @1.25:1[a]; C-FA 20% @ 1:1 replacement[a]		Silicone (Si), hot pour (hp), preform (pf) sealers	JPCP and CRCP
1993	≤0.487	Same		564	550 psi opening	FA 15% @ 1.25:1 replacement[a]		Si, hp, pf sealers	
1995	≤0.487	5%–8%		564	550 psi opening	AP #8 agg		2 saw cuts for T-joints; Si, hp, pf sealers	
1995 & 1999 (QA)	≤0.45	5.7%–7.3%; lot removal <4.5% or >9%; sublot 4%–10%		440	Flex ≥570 7d accept	PC/FA ≥ 3.2 (wt)[a] (aka 20% max)		Si, hp, pf sealer	
1999	≤0.487	5%–8%		564	550 psi opening	FA 15% @ 1.25:1 30% GGBFS[a]; AP #8 agg		L-jt single saw cut, #5–7[b] tie bars at 3′; T-jt 2 saw cuts	
2006–2012	≤0.487	5%–8%		564	550 psi opening	20% FA[a], 1.25:1 30% GGBFS[a], 1:1		same	60s mixing
2006 QC/QA	≤0.45	5.7%–8.9%; 2.5% max sublot range	Liquid membrane CC (M148)	440	7d Flex 570 accept, 550; opening	PC/FA ≥ 3.2 (wt)[a]; PC/ggbfs ≥ 2.3 (wt)[a]	WR type A, D, E, F, & G; AEA per AASHTO M154	**L-jt:** single saw cut, #5–7[b] tie bars at 3′; **T-jt:** 2-Saw cuts = T/3; 18′ max spacing; Si, hp, pf, elastomeric sealers	
2008 QC/QA	Same	Same	Same	440	Same	Same	Same	Same except no mention of D/3 after 2006	
2010 QC/QA	Same	Same	Same	400	Same	Same	Same	Same	
2012 QC/QA	Same	Same except min failed incr to 5.3%	Same	400	Same	Same	Same	L-jt: single saw cut, tie bars @ 3′, <9″ #5; >9″ #6	JT sealer: Si, hp, pf, elastomeric, polychloroprene

[a]Allowed Ap1-Oct15.
[b]Tie bar size for L-jt depends on pavement thickness: #5 @ <9″; #6 @ 9″-12″; #7 @ >2″ thick.
CC = Curing compound.
FA = Fly ash.
JRCP = Jointed reinforced concrete pavements.
JPCP = Jointed plain concrete pavements.
CRCP = Continuously reinforced concrete pavements.
GGBFS = Slag.
AEA = Air Entraining Agent.
QA = Quality assurance.
QC = Quality control.
= Specifications for mix designs controlled under QA and QC/QA practices.

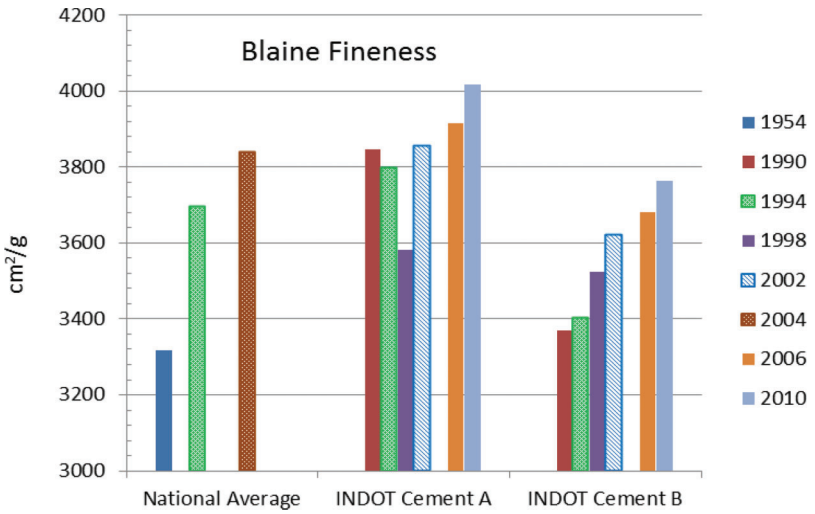


Figure 2.1 Blaine fineness of cements over the years.

TABLE 2.2
Average Blaine fineness values published for cements for years 1953–54, 1994 and 2004.

	Blaine cm^2/g from National Publications		
	1953–54	**1994**	**2004**
Mean	3316	3694	3840
STD	262	256	193
N	97	70	52
Max	4390	4210	
Min	2880	3000	

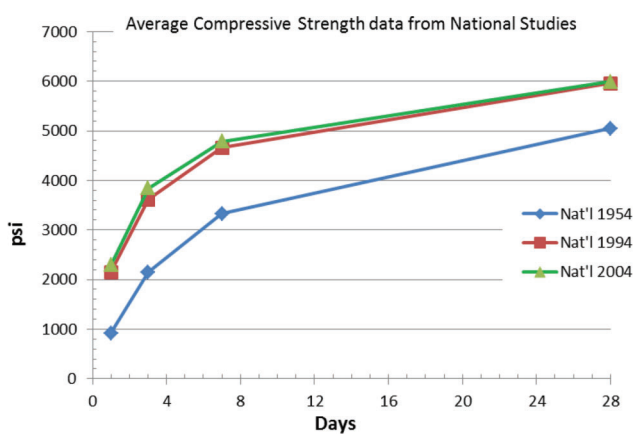


Figure 2.2 Average compressive strength development for cements from years 1953–54, 1994 and 2004.

The early-age strengths increased for cement B over the 20-year time span examined, going from the lowest 1-day and 3-day strengths in the early 1990s to the highest reported values in 2010. The 7-day strengths reported for 1990–2010 were all very similar for all data examined. The 28-day strength data fluctuate slightly for cement A but are similar to the national data, whereas the 28-day strengths for cement B appear to decrease with time.

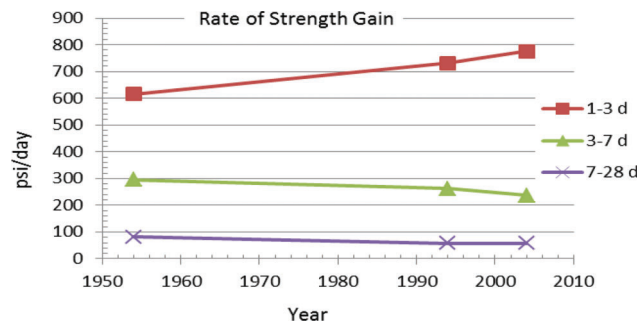
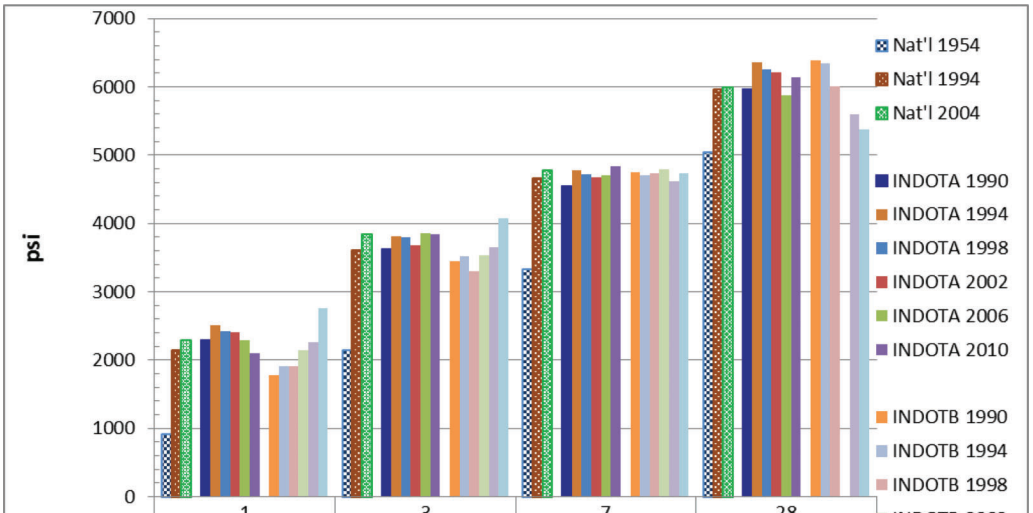


Figure 2.3 Changes of the rate of strength gain over the years of cements nationwide.

2.2.3 Chemistry

The cement chemistry can influence concrete properties such as strength and rate of strength gain. Generally dicalcium silicate (C_2S) hydrates more slowly than the tricalcium silicate (C_3S) therefore increased C_3S can contribute to higher early strengths. The national trend from 1954 to 2004 showed a decrease for the amount of C_2S, while the amount of C_3S increased (as shown in Figure 2.5). There was a slight decrease in C_3A between 1994 and 2004 with little change measured in C_4AF (Clifton & Mathey, 1971; Gebhardt, 1995; Tennis & Bhatty, 2005). The same amount of water is needed to hydrate C_2S and C_3S but C_3S produces twice as much calcium hydroxide (CH) then that formed when C_2S hydrates (Neville, 1997). (The implications of this are discussed further in Section 2.4.)

The changes in C_2S and C_3S in INDOT cements A and B since 1990 is not as clear of a trend as the average national data (as shown in Figure 2.6 and Figure 2.7). There is a distinct trend of decreasing C_2S for INDOT cement A, and the amount of C_2S in INDOT Cement B is lower in 2010 than previous years examined. Although the values for C_3S vary with time both INDOT cements had lower C_3S content in 1990 than in 2010.



	1	3	7	28
Nat'l 1954	915	2148	3331	5043
Nat'l 1994	2149	3612	4663	5959
Nat'l 2004	2292	3841	4786	5990
INDOTA 1990	2297	3625	4554	5977
INDOTA 1994	2508	3820	4784	6356
INDOTA 1998	2418	3793	4715	6250
INDOTA 2002	2404	3677	4673	6209
INDOTA 2006	2288	3856	4712	5881
INDOTA 2010	2103	3850	4841	6145
INDOTB 1990	1787	3453	4752	6394
INDOTB 1994	1915	3525	4700	6341
INDOTB 1998	1910	3306	4735	6013
INDOTB 2002	2143	3531	4799	
INDOTB 2006	2269	3648	4614	5595
INDOTB 2010	2760	4071	4741	5383

Figure 2.4 Strength data for INDOT cements A and B at 1, 3, 7 and 28 days.

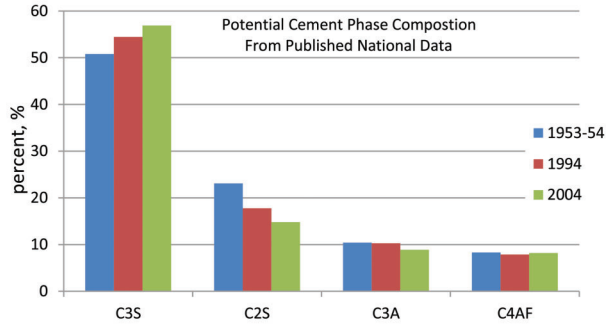


Figure 2.5 Average values for potential cement phase composition (from published data).

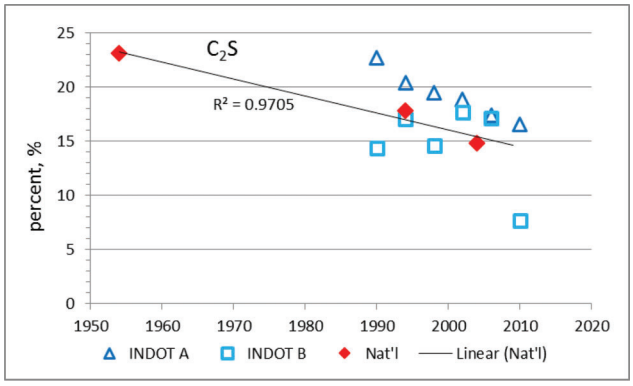


Figure 2.6 Changes in cement C_2S over time.

The sulfate content of cements also increased by nearly twice as much in the 50 years, from an average of 1.88% in 1954 to 3.26% in 2004 (as shown in Figure 2.8). This is not surprising because the cement fineness (Blaine) also increased over that time period, and as the fineness increases the C_3A becomes more readily available for early hydration and the amount of sulfate required to retard this quick hydration process and

avoid flash set increases (Neville, 1997). (The implications of this are discussed further in Section 2.4.) The slight decrease in C_3A in the more recent data also may be in response to the increased fineness in an effort to offset increased availability of C_3A (see Appendix A for more details). The sulfate content in Cements A and B varied with time but there is a general trend of increased sulfates from 1990 to 2010 with values in more recent years similar to each other and to the national average value. Figure A.1 (Appendix A) shows a correlation between increased fineness and increased percent sulfates.

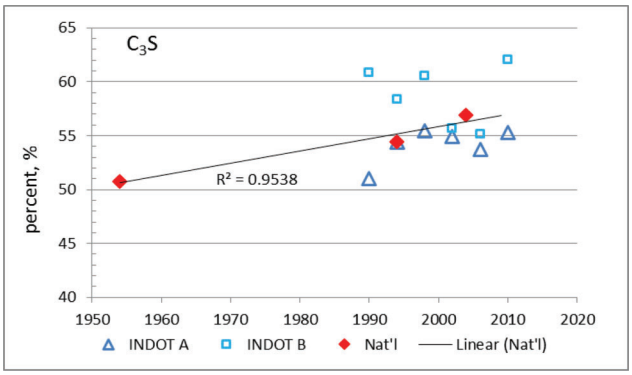


Figure 2.7 Changes in cement C_3S over time.

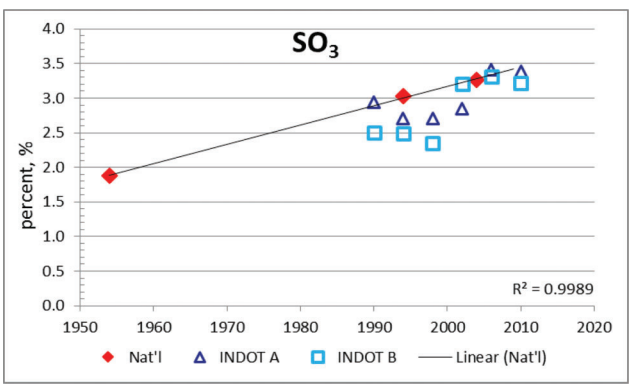


Figure 2.8 Changes in cement sulfate content over time.

2.3 Deicing Practices

The amount and types of deicers used by INDOT varies throughout the state. Information on the amounts and type of deicers used on a given pavement are not recorded, but usage within each district is available. A breakdown of usage from 2008–2010 by the six districts and subdistricts across Indiana are shown in Figure 2.9. (See figure legend for how the units of measure differ for different deicers). Additional deicer details are provided in Appendix A, Table A.5.

The primary deicer, by a very large margin, for all districts is sodium chloride (NaCl) primarily as a solid but also as a brine. The other deicers used included calcium chloride (CaCl) both solid and as a liquid solution, magnesium chloride (as a liquid solution) and a commercial product called Ice Ban. The amount that INDOT used of each of these deicers from 2008 to 2010 is given in Table 2.3.

It's not surprising that the northern districts of LaPorte and Fort Wayne, which generally have harsher, longer winters, had some of the highest deicer usages. Vincennes, the southern-most district used the least amount of deicers. Several details about deicer use are not recorded, such as (1) how much of what is used for anti-icing that may not become diluted if the storm event does not develop and (2) how much of which deicer is used on any particular pavement or pavement section. Information such as these would provide insight on how the deicers are affecting the concrete in the field and the pavement sections chosen to examine more closely.

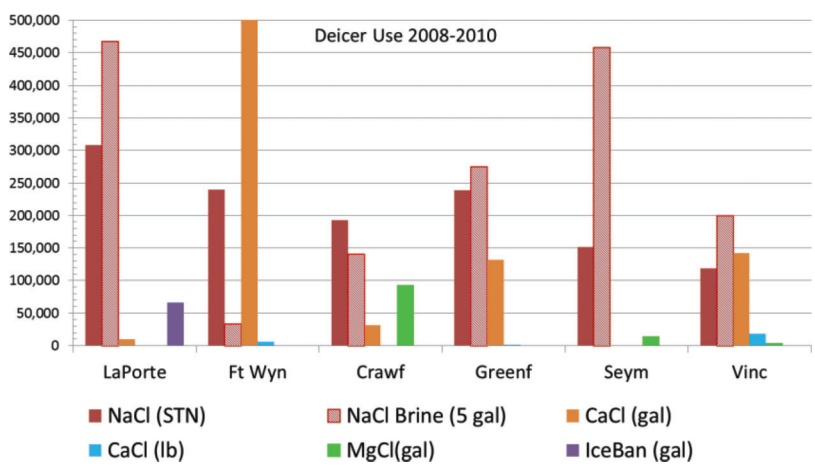


Figure 2.9 Summary of type and amount of deicer use by INDOT districts, 2008–2010. (Note: The unit of measures varies for different deicers. To compare solid to liquid NaCl, it could be estimated that a STN (ton) of NaCl can make nearly 1000 gals of brine. To develop this estimate the following assumptions and information from Wikipedia were used: brine can hold as much as 26% by weight NaCl at 0°C, brine has a density of 1.164 that of water if it contains 22% by weight NaCl; a ton of water is 240 gallons.)

TABLE 2.3
Deicers used by INDOT districts for 2008, 2009 and 2010.

	NaCl Brine (gal)	NaCl (STN)	MgCl (gal)	IceBan (gal)	CaCl (lb)	CaCl (gal)
LaPorte	2,340,280	307,817	0	66,176	0	9,556
Ft Wayne	173,819	239,619	0	0	5,680	501,715
Crawfordsville	705,154	192,723	92684	0	0	31,180
Greenfield	1,378,046	238,858	0	0	34	132,145
Seymour	2,293,198	151,851	14175	0	0	0
Vincennes	1,002,716	118,640	3500	0	17,835	142,092
Total	**7,893,212**	**1,249,508**	**110,359**	**66,176**	**23,549**	**816,687**

2.4 Summary and Discussion

Published national cement data and the information from the mill certificates on file at INDOT that were examined all show that the cement fineness, strength and chemistry have changed over the years. The fineness, measured as Blaine has increased for both the INDOT cements and cements across the country. The 1-day through 28-day strengths all have increased since 1954 as have the early-age rate of strength gains. The 7-day strengths appear to be the most consistent values across all cements examined since 1990.

The increase in both Blaine and C_3S contributes to the increase in 1-day and 3-day strengths since 1954. These changes in cement production are likely in response, at least in part, to demands from the concrete industry to increase production and reduce construction costs. The sulfates also have increased since 1954 to avoid set problems that can occur with finer cements and more readily available aluminates (Neville, 1997).

Some possible side-effects of these physical and chemical changes in cements to consider include:

- Increased development of early heat of hydration that could make it more susceptible to problems if concreting in high ambient temperatures
- Increased CH in the hardened concrete
- Increased sulfates in the hardened cements
- Reduced aluminates

Although increased CH in hardened concrete has been associated with long-term durability concerns, silicates from supplementary cementitious materials (SCM) such as fly ash and slag can combine with CH to form additional CSH (calcium silica hydrates) contributing to long-term strength and durability (Neville, 1997). Therefore the use of SCMs are especially valuable with modern cements in reducing the CH and increasing the CSH in the concrete and contributing to long-term durability.

Increased availability of sulfates can contribute to the formation of ettringite at inappropriate times during the life of the concrete. Although sulfate attack as ettringite forming in the cement paste matrix is not a common problem in pavement concrete, infilling of the concrete air void system with ettringite is common. Cracks can allow for increased ingress of fluids that can both change the pH and increase the transport of chemical constituents that can then be redeposited as secondary minerals, such ettringite in air voids.

Deicing salts can increase the moisture content of concrete leading to a higher level of saturation and susceptibility to freeze-thaw distress (Jones et al., 2013). Therefore the higher use of deicers combined with the exposure to more freeze-thaw cycles in the northern INDOT districts likely contribute to an increased susceptibility of concrete to freeze-thaw distress.

3. PAVEMENT SELECTION AND TESTING

3.1 Pavement Selection

Several concrete pavements were identified throughout Indiana by INDOT districts, the Study Advisory Committee (SAC) and the project team, and are listed in Table 3.1. Most of these pavements exhibited deterioration at the joints but some of them were reviewed as potential examples of non-deteriorated pavements. The project team visited all the concrete pavements that were under consideration. Where traffic allowed the team stopped to examine each pavement section more closely (i.e., non-interstate pavements). Photo-logs of these site visits were created from which the coring sites were selected by the project team and the SAC. The criteria considered for selecting the pavements to core included those representing concrete placed under various specifications, of different ages, exposed to different deicing chemicals, with joints showing various degrees of deterioration, with different joint sealers, placed on different subbase, with some construction or mix design information, with and without fly ash and accessible to the core rig.

Much of the construction and mix design information were not available for most of the pavements. The INDOT districts were surveyed to collect additional information which yielded a small amount of accurate data. A copy of the "State-wide Highway Planning Survey-Road Life Log" provided by the Seymour District (S) provided year built, location, pavement type and some configuration information for pavements being considered from that district (see Figure 3.1 for an example). The construction date was stamped on many of the Greenfield District pavements built from 1990. Between the records available and pavement site visits

TABLE 3.1
List of concrete pavements considered for further examination.

Pavement	Year Built	Comments	Cored
LaPorte District			
SR 933 South Bend	~1998?[a]	Cored and examined under SPR-3016	3016
I-94 Michigan City	1998-99[a]	Cored and examined under SPR-3016	3016
I-65 Merrillville, RP 248–256	12/01–11/02[b]	Minor D at T and L-jt (N of US 31)	
US 421 & SR 2 (MB)	??	T-jt and L-jt show D	
US 421 near I-90	~45 yrs old	District Survey Report	
Fort Wayne District			
I-469	??		
SR 19 Elkhart	??		
Greenfield District			
SR 38 New Castle near 25th St	8/15/90	EB no D, WB moderately severe D	X
SR 38 New Castle near Reddingdale Dr	6/14/90	Moderate D	X
SR 3 New Castle, RP 112–114	Older than1987	D common L-jt, patching every ~5 yrs	X
SR 3 New Castle, RP 115–117	1994	No D, good drainage	X
US 35, SR 28 & SR 3, N of Muncie	2003	Minor D	X
US 35 and Eaton St, near STA 170		~35 ft panels w/ mid-panel cracks	
Memorial Dr Muncie, IN	??	Probably D-cracking (near US 35 / SR 3)	
Indianapolis			
W 86th St near Payne Rd	~ 1998?[a]	Cored and examined under SPR-3016	3016
SR 67 ramp to EB lane of I-465	~ 1998?[a]	Cored and examined under SPR-3016	3016
I-65 and MLK Dr	~ 1998?[a]	Cored and examined under SPR-3016	3016
I-65 and I-70 RP 112.4–110-8 SB	1970s	Moderately severe L and T-jts	
I-70 and Ameriplex Pkwy	August 2004[b]	WB, some minor D, other problems	
I-465 Beltway (S to E)			
I-465 and 56th St	2003–04[b]	NB some D; SB little to no D	
I-465 & I-70	2003–04[b]	No D	
I-465 & Washington	1998[a]	No D	
I-465 & US 52	2000[b]	NB some D, SB no D	
I-465 & I-74	2001[b]	NB no D, SB significant D at CL	
I-465 & Emerson to I-74	1999[b]	No D	
I-465 & I-65 (±RP 106)	1995–96[a]	No D	
I-465 & US 31	1998[a]	WB no D, EB significant D at L-jts	
Town of Fishers, IN	1999–2005[b]	*INDOT Specifications followed*	
116th St, Phase 1, Fishers, IN	1999–2000[b]	Faulted, low steel, joints OK	
116th St, Phase 2, Fishers, IN	2003[b]	EB some L-jt D, WB good, no D	X
116th St, Phase 3, Fishers, IN	2004[b]	WB w/ L and T-jt D, EB good, no D	X
116th St, Phase 4, Fishers, IN	2005[b]	EB some D, WB no D, good	
Allisonville Rd, Phase 1	2001–02[b]	SB late season pour (D), NB no D	X
Allisonville Rd, Phases 2–3	2005+[a]	Good except some hand poured areas	
126th St EB	1997[a]	Good but replacing	
LaPorte District			
SR 933 South Bend	~1998?[a]	Cored and examined under SPR-3016	3016
I-94 Michigan City	1998–99[a]	Cored and examined under SPR-3016	3016
I-65 Merrillville, RP 248–256	12/01–11/02[b]	Minor D at T and L-jt (N of US 31)	
US 421 & SR 2 (MB)	??	T-jt and L-jt show D	
US 421 near I-90	~45 yrs old	District Survey Report	
Seymour District			
US 421, Madison, RP 0.73–0.99	1969	Slight to moderate D	X
SR 60, Clarksville, RP 60.33–61.3	2002[b]	No D (good)	
SR 250, Uniontown, RP 13.6–13.9	1960	Vlittle D, but some patches	X
SR 56 (RP 139.96–139.47)	"Older"	Heavily patched, probably D-cracked, difficult location to core	
US 50, Seymour, RP 104.4–105.6	1960	Patched, some D at some L-jt only	X
US 31, Seymour, RP 45.8–46.27	??	Vlittle D, some sm patches cracked	

[a]1995–1998 Specs.
[b]1999–2005 Specs.
D = joint deterioration.
L-jt = longitudinal joint.
T-jt = transverse joint.
Vlittle = very little.
"3016" refers to cores taken under SPR-3016 and results published in Arribas-Colón et al. (2012).

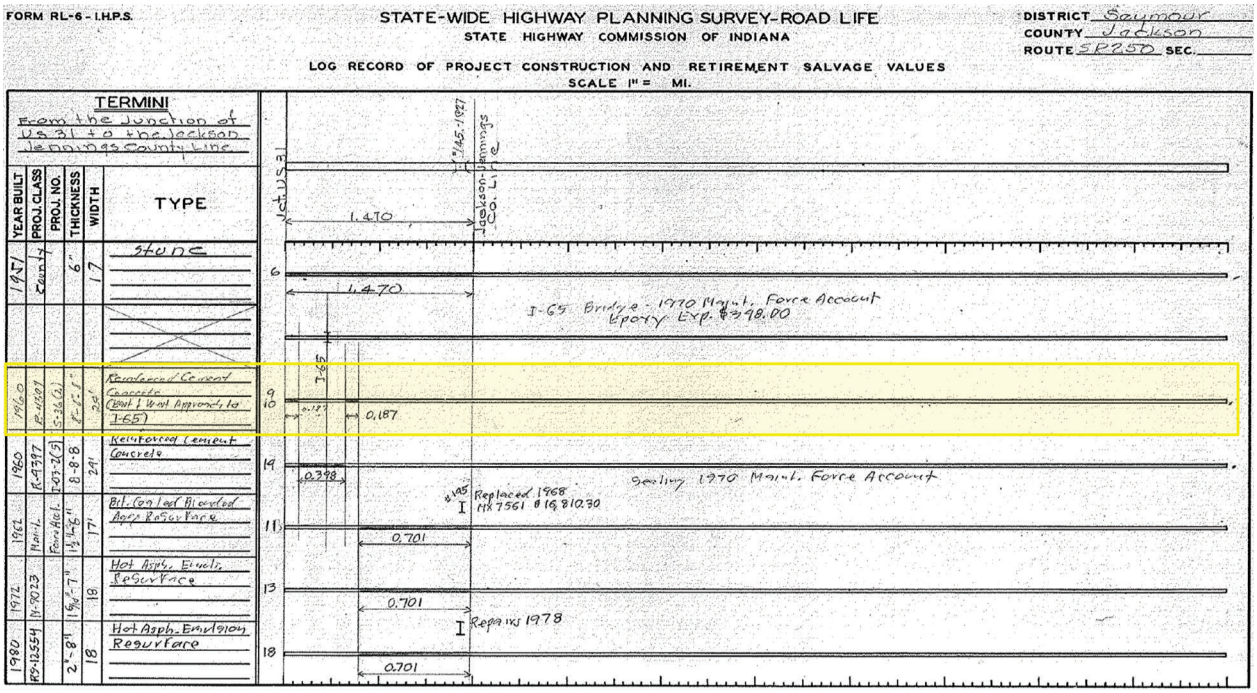


Figure 3.1 Pavement record provided for section of US 250 selected for coring (highlighted).

# cores	Phase	Stop	Location	Latitude	Longitude	Comments	Material and Contractor
	1		116th Street, Phase 1, EB			Very little steel used, no tie bars, no dowl baskets, cement treated subbase, skewed joints, faulted, check steel with locator (MIT Scan)	Builders, MicroAir AEA, gravel CA
			116th Street, Phase 1, WB				
2 (L & M)	2	1	116th Street, Phase 2, EB	39.956889	-85.978026	***EB shows deterioration*** along longitudinal joint only, transverse joint ok	2003: Berns & Berns: vinsol rosin
2 (L & M)		2	116th Street, Phase 2, WB	39.956889	-85.978026	WB is not showing any deterioration.	
3 (L, T, M)	3	3	116th Street, Phase 3, WB	39.959162	-85.945218	***WB w/ joint deterioration*** in both longitudinal and transverse joints.	2004: Berns & Builders: MicorAir AEA, Lehigh cement
3 (L, T, M)		4	116th Street, Phase 3, EB	39.959162	-85.945218	EB is in good condition	
	4	5	116th Street, Phase 4, EB	39.957742	-85.924312	***EB w/ deterioration at joints,*** whereas WB is in good condition	2005: Berns & Builders: MicorAir AEA, Lehigh cement
2 'good' (L & M), 2 'bad' (L & M)		6	Allisonville Road, Phase 1 (2001-2002)	39.955169	-86.040775	SB ***Both longitudinal and transverse joints show deterioration*** in front of Jiffy Lube - Late season pour, 'snow flying'. NB is not showing deterioration	Reith-Riley: Fly ash mix (fly ash from Rock Port)
		7	Allisonville Road, Phase 2 (2005)	39.939188	-86.051861	All joints are in very good condition. Minor jt deterioration in small areas that were hand poured.	
		8	Allisonville Road, Phase 3	39.92907	-86.062552	Joints are in good condition. The intersection with 96th St in very bad shape	
			Allisonville Road and 96th St			Hand poured, high w/cm, poor air void system, severe joint deterioration	
		9	126th Street, EB	39.971897	-85.999017	Good Joints	
		10	126th Street, EB	39.971518	-85.926294	Good Joints	

L = longitudinal joint
T = transverse joint
M= mid-panal

126th St (1997)	Joints in good condition but removing to reconstruct, widen, flatten...	Builders (Chris Tull)
126th E of I-69 (2010)	Joints in good condition after 2 winters, single saw cut, soy sealer	??

Projects in which MicroAir AEA was used.
Projects in which Vinsol Rosin AEA was used.

116th St Phases 2-4 followed FHWA and INDOT Specifications

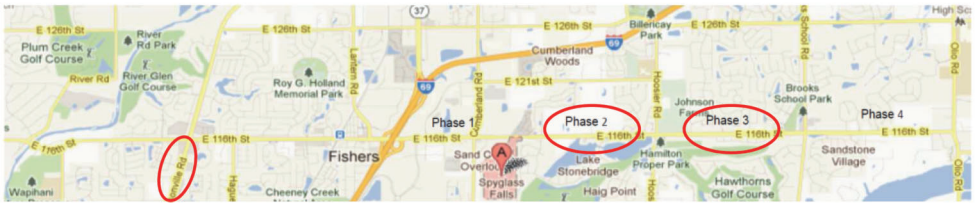


Figure 3.2 Summary of Town of Fishers pavement information provided by consultant (C. Tull, personal communication, 2011).

information such as year built, pavement thickness, joint spacing, pavement type and joint sealer were collected. Information available for pavements examined in the Town of Fishers was provided by CRT (C. Tull, personal communication, 2011). A summary of this information is shown in Figure 3.2.

Information not available was inferred based on the prevailing specification at the time of construction. As shown in Table 3.2 pavements were identified and selected to represent all major specification changes from 1960 to early 2000s. The pavements cored from Fishers represent concrete from 116th St, Phases 2 and 3. Both phases showed deterioration at the joints in one direction but not the other. Phase 2 was built with vinsol resin air entraining admixture (AEA) and Phase 3 was built with a synthetic AEA (MicroAir). The third section chosen for coring in Fishers was Allisonville Rd, Phase 1. This pavement showed deterioration only in the southbound (SB) lanes which was a fly ash mix poured late in the construction season during a snowfall.

The pavements selected for coring and further examination are presented in Table 3.2 alongside a summary of the concrete pavement specifications that were in place at the time of construction.

3.2 Database Development

A database was developed with the intention of documenting the pavement locations examined in this study and related information collected and developed. The concept was to provide a database with basic information so that queries and sorts could be performed such as looking for all concrete pavements that have air void analysis data, or all pavements constructed with fly ash before 1990. Links would be provided to any reports and test results related to cores taken from specific pavement locations. This was to be an active database so that all pavements and information collected under this project could be easily accessed at any future time, as well as information from previous projects and information from future projects added. The database would include mapping capabilities so that all pavement locations included in the database could be located whenever needed using precise latitude and longitude information. Pavement sections included in this database would appear on a map that had a hot link to the data.

Hotlinks to examples of these pages are as follows:

- Basic pavement locations and field information report page "Road Test": http://rebar.ecn.purdue.edu/jb/google/survey_form1.aspx
- Test results and specific core locations: http://rebar.ecn.purdue.edu/roadtest/coretestresults.aspx?corelocation=
- Maps of pavement locations: http://rebar.ecn.purdue.edu/roadtest/roadtest.html#

An example of the database Pavement Map is shown in Figure 3.3. Examples of the Pavement Location Information pages and the Core Test Results pages are shown in Appendix B.

These pages are available for populating with data and use, however the forms and process are somewhat clumsily and incomplete. It is recommended that a more complete and easy to use database be developed and maintained so that the information developed from this and other projects can be combined in searches and easily accessed for many years to come.

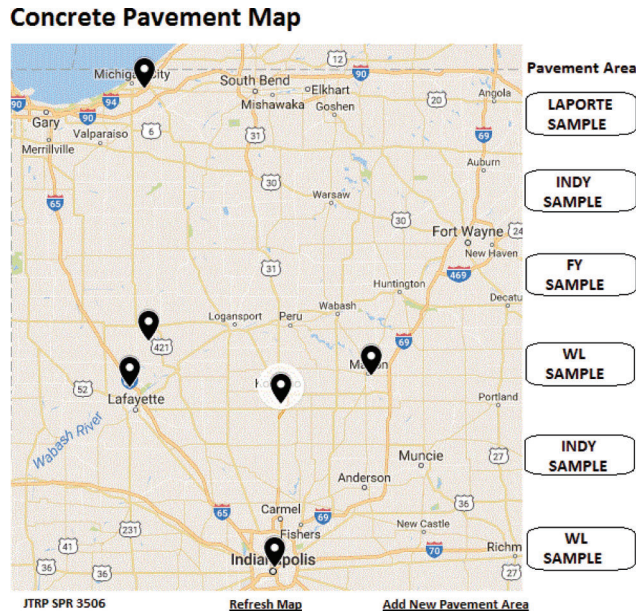


Figure 3.3 Example of the Database Pavement Locations Map.

3.3 Coring and Sampling

A total of 55 cores were retrieved from the pavement sections identified for sampling as shown in Table 3.3. Cores were retrieved from locations that showed deterioration at the joints and whenever possible, from nearby locations that showed no or noticeably less deterioration at the joints. A minimum of two 6″ diameter cores were taken from each location, one at the longitudinal joint and one near the mid-span of that same panel. If there was a visible difference in the pavement condition at the transverse joint then a core was taken at the transverse joint as well. After the core was removed additional in-place pavement conditions were examined including: pavement condition with depth; the condition of the saw cut, joint and joint sealer; and a rate of drainage or lack of drainage of the drilling water from the core hole. Figure 3.4 and Figure C.1 (Appendix C) explain how the unique core identifier describes the location of the core. On rare occasions, cores were obtained from patched concrete and/or vibrator trails. These cores were labeled P and V for patched concrete and vibrator trail respectively.

3.4 Test Specimens and Concrete Tests

Cores were photographed, examined megascopically and features noted such as aggregate type, base type, condition of joint face and joint sealer, cracks and pavement thickness. Several cores were cut and sectioned into specimens for testing and further microscopic examination.

3.4.1 Lab Test

Permeability and FT resistance of select pavement specimens were measured. Permeability was measured

TABLE 3.2
Pavements cored compared to prevailing specifications at the time of construction.

Pavements Cored	Year	Spec	Cement (min)	Fly Ash	w/cm	Strength	% Air	L-jt	Curing	T-jt	WR	AEA	Agg.	Mixing	Jt Filler
	1952	D1	564		0.487		3–6	Hand tool, etc.	Wet cure 96 hrs	Dowels, keyways					
	1957	D1	564		0.487		4–7[a]	Hand tool, etc.		Dowels, keyways		T12 vinsol resin[a]			
S: SR 250, US 50[b,c]	1960							Same as 1957							
	1963	D1	564		0.487	550 (O)	4–7	Sawn poss.[a]	White pigment poss.[a]	Sawn poss.[a]		T12 vinsol resin	#2 or #5[a]	50 sec[a]	
S: US 421[b,c]	1969	501	564		0.487	550 (O)	5–9	Sawn poss.			Y[a]		#2 or #5	50 sec	905
G: SR 3 Newcastle, pre-1987[c,d]	1971			Same			5–8[a]				Same				
	1974						Same						#5 or #8	Same	
	1978							All same as 1974							
	1985	501	564	1P[a,e]	0.487	550 (O)	5–8				Y		#8[a]		No Si mention
G: SR 38 Newcastle[b,c]	1988	501	564	15% F @1.25:1 20%C @1:1[a]	0.487	550 (O)	5–8				Y		#8		Si[a], h/p pf[a]
G: SR 3 Newcastle[c,f]	1993	501	564	15% @ 1.25:1[a]	0.487	550 (O)	5–8				Y		#8		Si, h/p pf
L: I-94 Michigan City, SPR-3016[b]	1995	501	564	15% @ 1.25:1	0.487	550 (O)	5–8			2 saw cuts[a]					Si, h/p pf
		502 QA[a]	440[a]	Min PC/FA = 3.2 by wt[a]	0.45[a]	570 (A)[a]	5.7–7.3[a]						AP[a] #8		Si, h/p pf

Continued

TABLE 3.2
(Continued)

Pavements Cored	Year	Spec	Cement (min)	Fly Ash	w/cm	Strength	% Air	L-jt	Curing	T-jt	WR	AEA	Agg.	Mixing	Jt Filler
Town of Fishers[b,c] US 35 N Muncie[e,f]	1999	501	L-jt: Single saw cut allowed and tie bar spacing specified[h]; otherwise same as 1995												
		502 QA	Same as 1995												
	2006	501 QC/QA[a]	440[a,g]	Min PC/FA = 3.2 by wt[a]	0.45	550 (O) 570 (A)	5.7–8.9[a]	Single saw cut[h]	Liq membrane forming M148[a]	18′ max spc 2 saw cuts T/3 (2005)[a]	A, D, E, F, G[a]	AASHTO M 154[a]	AP #8		Si, h/p pf, el
		502	564	20% @1.25:1	0.487		5–8						AP #8	60 sec[a]	
	2008	Same as 2006													
	2010	501 QC/QA	400[a,g]	Min cement reduced to 400 lbs; otherwise same as 2006 501 QC/QA Spec											
		502	502 Spec same as 2006												
	2012	501 QC/QA	<5.3% air fails, #7 tie bar dropped, and poly allowed as joint filler; otherwise same as 2010 501 QC/QA Spec												
		502	502 Spec same as 2006												

[a]When Spec first appears.
[b]Mod or varying degree of Jt deterioration.
[c]Cored under this project.
[d]Major Jt deterioration.
[e]Fly ash allowed from 4/1 to 10/15.
[f]Little to no Jt deterioration.
[g]Cement content for Concrete Mix Designs (CMD) for submittal (S) and approved for production (P).
[h]L-jt tie bars @ 3′spacing: <9″ #5; 9″-12″ #6; >12″ #7; in 2012 <9″ #5 and >9″ #6.
(O) = Opening strength.
(A) = Acceptance strength.
L-jt all T/3: T-jt T/4 until 2006 changed to D/3.
Jt filler: Si = silicone, h/p = hot pour, pf = preformed, el =elastomeric, poly = polychloropren.

using ASTM C1202 Rapid Chloride Permeability Test, and FT resistance was measured using cyclical freezing and thawing similar to ASTM C666 Procedure A (freeze and thaw in water) except test specimens were 4″ diameter by 1″ thick disks instead of the recommended prisms.

3.4.2 Microscopic Examination

Optical microscopy. The hardened air void system was characterized using ASTM C457 Modified Point Count Method using 75x magnifications. Further analysis included microscopic examination of the polished slab at magnifications up to 120X. Additional information collected included, infilling of air voids, existence and location of microcracks.

Scanning Electron Microscope (SEM). Select specimens were examined under the Scanning Electron Microscope (SEM). The purpose of this task was to determine more detailed features of the concrete and identify any structural or chemical changes to the paste. Some features observed included the presence of fly ash, the degree of unhydrated cement, infilling of the air voids and composition of infilling, deposition of secondary minerals in the paste, presence of chlorides, microcracks, dissolution/leaching, carbonation, mineralogy of the coarse aggregate, and aggregate or paste distress.

4. TEST RESULTS AND ANALYSIS

4.1 Field and Megascopic Observations

Different degrees of deterioration were observed during the detailed visits to 21 pavement sites. For this report the different degrees of joint deterioration were classified into five levels of severity: (a) sound joints with no deterioration, (b) minor deterioration, (c) moderate deterioration, (d) moderately severe deterioration and (e) severe deterioration. Photographs showing examples of these degrees of deterioration are shown in Figures C.2 through C.6, Appendix C.

The 11 pavement sections cored represent over 40 years of concrete paving in Indiana, from 1960 to 2004. These pavements were selected with the expectation that they represent the design, materials, construction practices and INDOT specifications for concrete pavements that were prevalent at the time of construction.

Table 4.1 summarizes some of the pavement characteristics identified during the field site visits and megascopic examinations of the cores. Although the coarse aggregates (CA) were primarily quarried carbonate aggregates three pavements had gravel as the

TABLE 3.3
Lists of core locations.

Core Locations	Deteriorated Joint	Non/less Deteriorated Joint	Core ID	Comments
Greenfield (G)				
US 35 near SR 3	A, C, D	E, F, Z	G-US35	
SR 38 and Reddingdale Dr,	A, C, D	E, F, Z	G-SR38-R	
SR 38 near 25th St	A, C, D	E	G-SR38-25	
SR 3, New Castle, RP 113–115	A, C, D		G-US3-113	S of SR 38,
SR 3 New Castle, RP 115–117		E, F, Z	G-US3-115	N of SR 38
Seymour (S)				
US 421, Madison, RP 0.73–0.99	A, C, D	E, F, Z, V	S-US421	
US 50, Seymour	C, D, PC, PD	F, Z	S-US50	
SR 250, near I-65 & Uniontown	A, C, D	E, F, Z	S-SR250	
Fishers (F)				
116th St, Phase 2	C, D	F, Z	F-116-P2	
116th St, Phase 3	A, C, D	E, F, D	F-116-P3	
Allisonville Rd, Phase 1	C, D	F, Z	F-AV-P1	

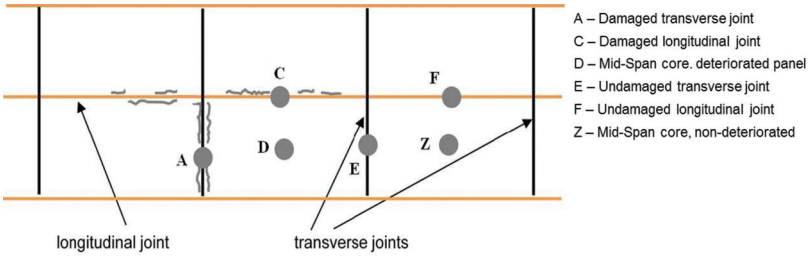


Figure 3.4 Core locations reference used in the identifier nomenclature.

TABLE 4.1
Summary of megascopic pavement characteristics.

Core ID Designator	Yr Built	Pvmt Thickness (in)	CA Type	CA Top Size (in)	Saw Cut (in)	Joint Sealer	Comments
G-SR38-R	1990	10.5	Gravel	1		Si w/br	Pop-outs
G-SR38-25	1990	10.5	Gravel	1		Si w/br	Pop-outs
G-SR3-113	Pre-1987	9	Qcarb	1.5	2.5	Si w/br	Pop-outs
G-SR3-115	1994	10.5	Gravel	1.5	2.5	Si w/br	Mid-panel cracks common
G-US35	2003	12.2	Qcarb			Si w/br	
S-US421	1969/1996?	12.75	Qcarb	1	3.5	Si w/br	
S-US50	1960		Qcarb		2.5	Hot pour	
S-SR250	1960		Qcarb			Hot pour	
F-116-P2	2003					Si w/br	
F-116-P3	2004					Si w/br	
F-AV-P1	2001-02					Si w/br	

Qcarb = quarried carbonate.
Si w/br = silcone w/backer rod.

coarse aggregate, all built in the early 1990s. The surface of the three pavements that were built between 1980 and 1990 were pock-marked with pop-outs, two had a gravel CA and one had a quarried carbonate CA. Pop-outs suggest the aggregate had some sensitivity to expansion caused by moisture and/or freezing and thawing.

The oldest pavements examined in this study were two pavements built in 1960 in the Seymour District. These oldest pavements were unique in that both were the only jointed reinforced concrete pavements (JRCP) examined, and the only pavement that used hot pour as the joint sealer (no backer-rod) (as shown in Table 4.2). Although maintenance has been required over the years it appears to be more related to mid-panel cracks and the long panel lengths. The original joints in these 50+ year old pavements appear to have only minor to moderate degrees of deterioration, whereas the joints in some of the more recent patches are in worse shape than the original older concrete (as shown in Figure 4.1.) Additional photographs and descriptions of the Seymour District pavements can be found in Appendix C.

Drainage of the subbase was estimated by observing the rate at which the water drained from the core hole after drilling (as shown in Figure 4.2). Therefore this qualitative drainage is more a measure of the base capacity to move water through and does not reflect the ability of the joint to move water.

All pavements were built on some type of granular base and therefore expected to have adequate drainage. However the rate that the drilling water drained varied greatly from pavement to pavement and often varied from joint compared to mid-panel in the same panel. Occasionally the drainage could not be observed because of limited traffic control and safety reasons.

There appears to be a link between how drainable the base is and the degree of joint deterioration observed, although it is not a one-to-one correlation (as shown in Figure 4.3). Based on the 12 pavement sections in which drainage information was collected the following was observed:

- 10 of the pavement sections had good drainage at the mid-panel.
- Only 5 pavement sections had good drainage at the joints.
- All 6 pavements with no or minor joint deterioration had moderate to good drainage at joints (as highlighted with a red circle in Figure 4.3).
- All 3 pavements with moderately slow to slow drainage at the joints had moderate to severe joint deterioration (as highlighted by a blue circle in Figure 4.3).
- Some pavements (3) that had moderate joint deterioration still had good drainage at the joints.

The drainage of the base is slower at the joints than at the mid-panel for half the pavements examined. It is uncertain what initially caused the base to drain slower at some joints, whether the joint deterioration contributed to slowing the drainage or the slower drainage contributed to increased joint deterioration. It is apparent that the pavements with the slowest drainage had the worst joint deterioration suggesting that the phenomena are related.

It is easy to imagine that as the concrete at a joint deteriorates it may allow more debris to wash through the joint and into the base contributing fines that may reduce the hydraulic conductivity of the base. Whereas, if the drainability of the base decreases then the amount of moisture retained in the concrete at the joint may increase enabling the concrete to more readily reach a state of supersaturation rendering it more susceptible to FT distress. Further work is needed to better understand the initiating mechanism(s) and exactly how and under what conditions the joint deterioration and base drainage relate in order to devise better strategies for maintenance, mitigation and building durable joints.

4.2 Optical Microscopy and Air Void Analysis

ASTM C457-2012 (*Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete*) were followed using a reflective light stereo microscope set at 75X magnification to determine the total air content, spacing factor and specific surface area. Although INDOT

TABLE 4.2
Drainage, joint conditions and additional field observations.

District	Highway	Year Built	Base	Jt Sealer	Level of Joint Deterioration	Drainage		Comments
						Joint	Mid-panel	
Greenfield	**SR 38** – Reddingdale Dr	1990	Granular,	Si-w/br	Minor	Good	Good	
	SR 38 – 25th St Deteriorated sections	1990	Granular w/crushed stone	Si-w/br	Severe	Slow-mod	Good	Curb/gutter; water in deteriorated joints
	Good sections				Minor	Mod-good	Mod-good	
	SR 3 – MP113	"Pre-1987"	Granular, coarse sand	Si-w/br	Moderately-severe, L-jt and T-jt	Slow	Good	Sealer missing at many deteriorated joints: water in deteriorated joints; many T-jts patched
	SR 3 – MP 115	1994	Crushed stone	Si-w/br	None	Moderate	Good	3/8″ jts, mid-panel cracks common
	US 35 – at SR 28 & SR 3	2004		Si-w/br	Minor	Moderate	Good	
Seymour	**US 421**	1969?	Granular	Si-w/br	Minor – Mod	Good	Good	Pvmt tined, appears newer than 1969
	SR 250 – E of I-65	1960 JRCP	Sandy	HP	Orig joints minor; jts in patches – mod			Original pvmt has 35′ jt spacing, in good condition.
	US 50 – Tipton St in Seymour	1960? JRCP	Crushed stone w/ fine sand	HP	Moderate, but heavily patched	Good	Good	Several generations of patches
Fishers	**Fishers** – 116th St, Phase 2	2003	Crushed stone	Si-w/br	EB: L-jt mod; WB & EB T-jt: none		Good	Si sealer removed at some joints
	Fishers – 116th St, Phase 3	2004	Crushed stone	Si-w/br	WB: mod	WB: slow	WB: slow	
					EB: none	EB: mod	EB: good	
	Fishers – Allisonville Rd, Phase 1	2001–2002	Crushed stone	Si-w/br	SB: mod NB: none	Good	Good	SB paved "while snowing"

Si_w/br = silicone sealer with backer rod
HP = hot pour
Qcarb = quarried carbonate
G = gravel
CA = coarse aggregate
Mod = moderate
Vgood = very good

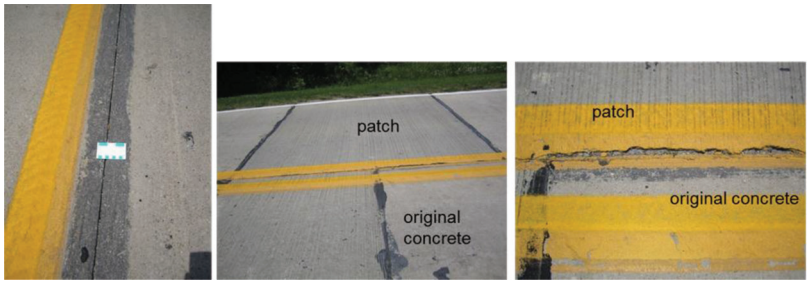


Figure 4.1 US 50 pavement in Seymour District showing original 1960 pavement and more recent patches.

Figure 4.2 Drainage from the core holes during drilling varied greatly, from good (left) to moderate (center) to slow (right).

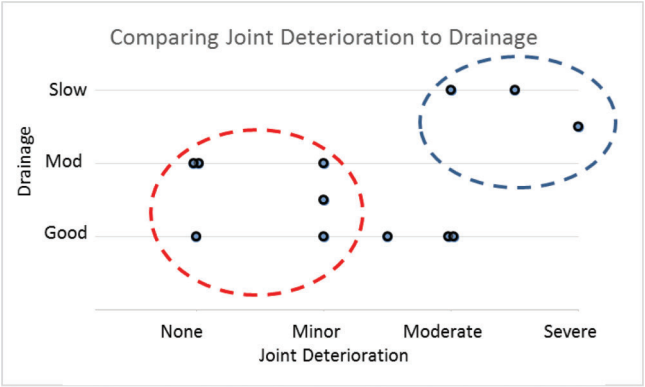


Figure 4.3 Comparing drainage of the base at the joint to concrete deterioration at the joint.

specifications for total air content has changed over the years, the pavements cored from the Greenfield District were built under the specification that required 5% to 8% air in the plastic concrete at the time of placement. The Portland Cement Association (PCA) suggests that concrete pavements exposed to extreme freeze-thaw environments should have an entrained air system with a spacing factor of ≤0.008 in and a specific surface of ≥600 in^2/in^3 (Kosmatka et al., 2002). However, at this time there is no easy, accurate method to measure these parameters in fresh concrete at the time of placement. Measurements of the hardened air void system after years of service (aka existing air void system) may show reduced air void parameters compared to what the "as constructed" air void system was, due to infilling of secondary minerals.

As examined, the concrete from the 15 cores tested, had an existing air systems that did not satisfy all three criteria recommended for freeze-thaw durability (as shown in Table 4.3). Comparing the air void parameters in the all cores for each pavement the following observations were made:

- US 35 showed an unexpected trend. Even though the total percentage of measured air was low in every core (ranging from 2.5% to 4.3%), all met or nearly met the specific surface criteria. Oddly the only concrete with an acceptable spacing factor was from a deteriorated joint showing minor deterioration.
- SR38-R panels without deteriorated joints had a better measured air void system than the panels with deteriorated joints. All cores had at least one parameter that passed the criteria. As expected, considering the higher

potential for water ingress and subsequent infilling of voids near the joint, better spacing factors were observed at mid-panel cores then at joints.

- SR38-25 panel with less deterioration at the joints had a better measured air void system than the panel with deteriorated joints but neither were good.
- SR3 panels had measured air void systems parameters that were similar in panels with and without joint deterioration and none met the given criteria for durable concrete.
- In any given panel the mid-panel core had a better measured air void system than that of the core(s) taken at the joint(s).

4.3 Scanning Electron Microscopy

A scanning electron microscope (SEM) equipped with energy dispersive x-ray (EDX) capabilities was used to examine the concrete microstructure. Small concrete samples (1 in^3) that represent different depths from the pavement surface were cut from the cores and polished. Table 4.4, Table 4.5, Table 4.6, and Table 4.7 summarize different features such as fly ash, and the occurrence of Friedel's salt, ettringite, and unhydrated cement particles that were detected during SEM analysis of samples taken from these Greenfield District cores.

US35 samples contained fly ash, which was not surprising since it was built in 2004 when specifications allowed for the regular use of fly ash in concrete pavements. Friedel's salt, a calcium chloroaluminate was observed in mid-panel cores up to 2″ deep and in joint cores up to 5″ deep (as shown in Figure 4.4) (concrete below 5″ was not examined). Many air voids were filled with ettringite (as shown in Figure 4.5) and the degree of infilling increased with depth from the pavement surface. Ettringite was more prevalent in joint cores compared to mid-panel cores suggesting

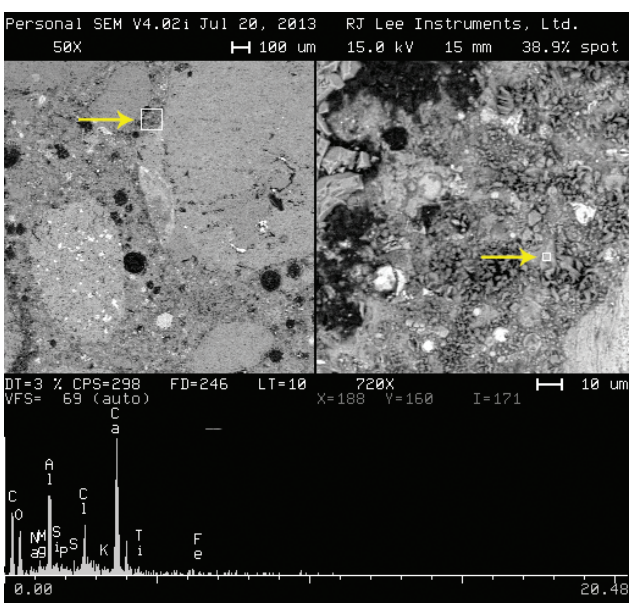


Figure 4.4 Friedel's salt. The arrow on the left image points to the area that is shown on the right at higher magnification. The arrow on the right points to the area scanned for the elemental analysis.

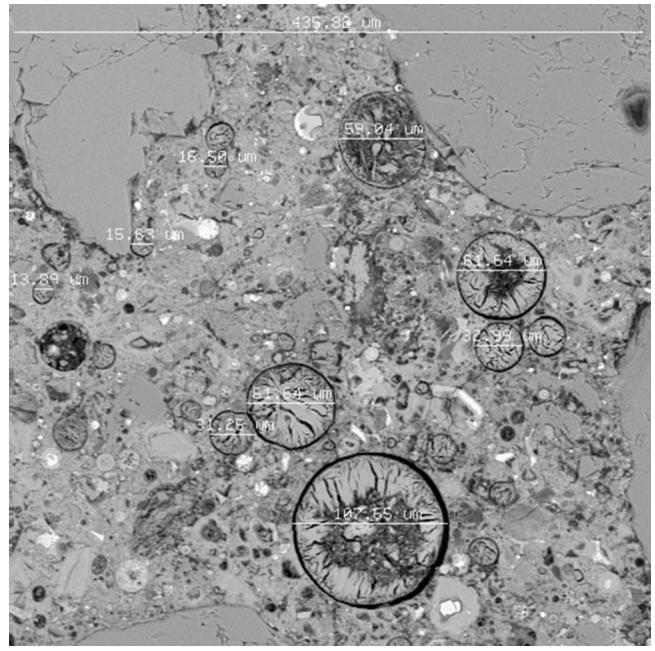


Figure 4.5 Ettringite infilling the air voids.

increased saturation and movement of moisture in the concrete at the joints.

SR38-R concrete samples from the relatively less deteriorated joints contained fly ash, but no fly ash was detected in cores extracted from the panel exhibiting more deterioration at the joints. Similar to what was observed for US35 core samples, the amount of ettringite lining and filling the air voids in the concrete from SR 38 also increased with depth, with the infilling more predominant for joint cores than mid panel cores. Also, a higher degree of infilling was observed for deteriorated joint core without fly ash (SR38-R-C) compared to concrete from well performing joints that contained fly ash (SR38-R-E). Friedel's salt was detected up to a depth of 5″ for joint cores but only up to 2″ for cores obtained from mid-panel locations. The amount of Friedel's salt detected in the joint cores was generally constant from 0″ to 3″ from the top of the pavement which related to the saw cut depth. The amount of Friedel's salt then decreased gradually with increasing depth. Figure 4.6 shows a Friedel's salt deposit in the sample obtained from SR 38.

SR38-25 concrete exhibited increased ettringite infilling air voids with depth from the pavement surface, similar to the pattern found in US35 and SR38-R concrete cores. Also, a higher degree of infilling was observed for joint cores compared to mid panel cores. The chlorides, detected primarily as Friedel's salt, were encountered less frequently with depth. Another interesting trend observed in the SR38-25 samples was that calcium hydroxide (CH) deposits were encountered more frequently with depth. For example, greater amounts of CH were seen in sample C–4 compared to C–3 and E–3 had more CH deposits than E–1 (again samples ranged from depths of 0 to 4″).

TABLE 4.3
Results for air void analysis, freeze-thaw and conductivity tests.

Core Number Greenfield District	Air Void Analysis – ASTM C 457				Infilled Air Voids				FT Test		Comment
	% Air[a]	Spacing Factor (in)	Specific Surface (in^{-1})	% Paste	Optical 75x	SEM >400x	Conductivity S/cm	RCP Coulombs	Δ Mod Elasticity (%)	Cycles to Fail	
TARGET VALUES	***5–8%***	***≤0.008***	***≥600***								
G-US35-A2	2.5%	0.015	657[b]	23.9%	****				42.6/16.7		Low air w/ high infilling; not enough mat'l for all tests
G-US35- C1	3.9%	0.008[b]	727[b]	24.0%	*	*to****			41.3/N.A.		Low % air with no infilling (optical); high infilling at 3–5″ (SEM)
G-US35-D1	4.3%	0.015	604[b]	22.9%	*	*to***				189	Marginal total air; no infilling (optical); infilling noted at 3–4″ (SEM)
G-US35-E1						*to****			40.8/30.6		Not enough mat'l for all tests; may have induced cracks during core retrieval
G-US35-F1	2.9%	0.013	593	23.1%	**	*to****					Low air content with some infilling worse at 3–5″ (SEM)
G-US35-Z1	2.9%	0.014	590	23.8%	*	*to***			37.3		Low air content with no infilling (optical), moderate infilling at 3–4″ (SEM)
G-SR38-R-C1	3.5%	0.0097	851[b]	23.4%	***	**to****	0.6164	2952			
G-SR38-R-D1	4.9%	0.0078[b]	538	20.0%	*	*to****	0.2397	2766	28.4		Increased infilling with depth (SEM)
G-SR38-R-E1	4.0%	0.0084	747[b]	24.3%	*	*to****	0.2344	5846		95/240	Infilling high 3–5″ (SEM); 2 FT samples tested
G-SR38-R-F1	5.5%[b]	0.0087	521	23.9%	*					120/95	2 FT samples tested
G-SR38-R-Z1	3.6%	0.0052[b]	770[b]	23.4%	*	*to***	1.0777	537	42		Infilling increased with depth (SEM)
G-SR38-25-A1							0.7541	1586		95/246	Top of core not obtained; 2 FT samples tested
G-SR38-25-C1	4.0%	0.0261	235	24.5%			0.5576	821		205	
G-SR38-25-D1	4.4%	0.0156	345	23.9%			0.4055	757			
G-SR38-25-E2	4.3%	0.0167	329	23.4%			0.1611	2978		167/300	2 FT samples tested
G-SR3-113-D1	4.2%	0.0098	510	21.2%	*	*to***			31.8		
G-SR3-115-E1	4.4%	0.0093	431	23.2%	**	*to****				95/61	
G-SR3-115-F1	4.1%	0.0132	397	21.6%	*					61/61	Higher % entrapped air
G-SR3-115-Z1	4.7%	0.0088	542	22.6%	*	*to***			30.8		

[a]% air is observed air and may not reflect original entrained air void system.
[b]Meets the specified criteria.
G-SR38-R-A1 and G-SR3-113-AE1: Partial cores; not enough material (mat'l) for testing.
Occurrence of Infilled air voids: *Vlow-none; **low; ***moderate; ****high.

TABLE 4.4
SEM analysis results for US35 cores.

Core	Distance[a]	Joint	Unhydrated Cement[b]	Fly Ash	Ettringite Infilling Air Voids	Chlorides[c]
C	0–1	Yes	H	Yes	*	H
	1–2		M		*	L
	3–4		VL		**	L
	4–5		VL		****	VL
D	0–1	No	VL	Yes	*	NA
	1–2		VL		*	VL
	3–4		VL		***	NA
E	0–1	Yes	M	Yes	*	H
	1–2		M		*	H
	3–4		VL		***	L
	4–5		VL		****	L
F	0–1	Yes	M	Yes	*	H
	1–2		M		*	H
	3–4		VL		****	L
	4–5		VL		****	VL
Z	0–1	No	VL	Yes	*	VL
	1–2		VL		*	VL
	3–4		VL		***	NA

[a]Distance of the sample from the top of the pavement surface (in inches).
[b]Frequency at which the feature is encountered; VL: very low; L: low; M: moderate; H: high; VH: very high.
[c]Chlorides detected as Friedel's salt and chlorides bound to CSH.
Ettringite infilling air voids: *Vlow-none; **low: ***moderate; ****high.

TABLE 4.5
SEM analysis results for SR38-R cores.

Core	Distance[a]	Joint	Unhydrated Cement[b]	Fly Ash	Ettringite Infilling Air Voids	Chlorides[c]
R-C	0–1	Yes	H	N	**	H
	1–2		H		***	H
	3–4		VL		****	M
	4–5		VL		****	L
R-D	0–1	No	L	N	*	L
	1–2		VL		**	M
	3–4		VL		****	NA
R-E	0–1	Yes	H	Y	*	H
	1–2		M		**	VH
	3–4		VL		****	M
	4–5		VL		****	L
R-Z	0–1	No	VL	Y	*	L
	1–2		VL		**	M
	3–4		VL		***	NA

[a]Distance of the sample from the top of the pavement surface (in inches).
[b]Frequency at which the feature is encountered; VL: very low; L: low; M: moderate; H: high; VH: very high.
[c]Chlorides detected as Friedel's salt and chlorides bound to CSH.
Ettringite infilling air voids: *Vlow-none; **low: ***moderate; ****high.

SR3 cores were analyzed using SEM, and similar to what was observed in samples from US 35 and SR 38, the amount of ettringite observed lining and filling the air voids increased with depth and was more prominent in concrete near the joint (SR3-E) compared to mid panel core (SR3-Z). Friedel's salt was detected in samples obtained from the top 2″ of the cores only and more abundant near the joint. Fly ash was detected in the newer concrete placed in 1994 (SR3-115) as well as the older concrete placed prior to 1987 (SR3-113). Of the parameters examined, the older age of the concrete, and therefore exposure to more freezing and thawing cycles at SR3-113 compared to SR3-115 may be a key reason for the more severe joint

TABLE 4.6
SEM analysis results for SR38-25 cores.

Core	Distance[a]	Joint	Unhydrated Cement[b]	Fly Ash	Ettringite Infilling Air Voids	Chlorides[c]
25-A	1–2	Yes	VL	Yes	***	M
	2–3		VL		****	M
25-C	0–1	Yes	L	Yes	*	M
	1–2		VL		***	H
	3–4		VL		****	L
	4–5		VL		****	VL
25-D	0–1	No	VL	Yes	*	M
	4–5		VL		***	VL
25-E	0–1	Yes	VL	Yes	**	H
	1–2		VL		***	H
	3–4		VL		****	M

[a]Distance of the sample from the top of the pavement surface (in inches).
[b]Frequency at which the feature is encountered; VL: very low; L: low; M: moderate; H: high; VH: very high.
[c]Chlorides detected as Friedel's salt and chlorides bound to CSH.
Ettringite infilling air voids: *Vlow-none; **low: ***moderate; ****high.

TABLE 4.7
SEM analysis results for SR3 cores.

Core	Distance[a]	Joint	Unhydrated Cement[b]	Fly Ash	Ettringite Infilling Air Voids	Chlorides[c]
113-D	0–1	No	VL	Yes	*	L
	5–6		VL		***	L
115-E	0–1	Yes	VL	Yes	*	M
	1–2		VL		***	H
	5–6		VL		****	L
115-Z	0–1	No	VL	Yes	*	L
	1–2		VL		**	L
	5–6		VL		***	VL

[a]Distance of the sample from the top of the pavement surface (in inches).
[b]Frequency at which the feature is encountered; VL: very low; L: low; M: moderate; H: high; VH: very high.
[c]Chlorides detected as Friedel's salt and chlorides bound to CSH.
Ettringite infilling air voids: *Vlow-none; **low: ***moderate; ****high.

deterioration since air void parameters and SEM results were comparable.

Summarizing the SEM analysis for all cores examined, the degree of ettringite lining and filling the air voids increased with depth from the top of the pavement surface to approximately 5″ below the surface. A higher degree of infilling of air voids was observed in the joint cores compared to mid panel cores. The unhydrated cement grains were more common near the joint face within the top 2″ of the pavement surface. Unhydrated cement grains in hardened concrete relate to poor curing. The low occurrence of unhydrated cement grains near the pavement surface in mid-panel cores suggests that the curing of the bulk of the concrete was adequate. Yet less than optimum curing of the concrete in the top 2″ of the joint face appears to be a common problem (as shown in Tables 4.4 and 4.5).

4.4 Freeze-Thaw Durability

Freeze-thaw (FT) testing was performed on 4″ diameter by 1″ thick disks using a FT cycling method similar to ASTM C 666-2003 procedure A (freeze and thaw in water). One disk was extracted from the mid panel core, and two disks were obtained from the joint core whenever possible; one sample as close to the joint face as practical (FT1) and one approximately an inch away from the joint face (FT 2) (as shown in Figure 4.7). The procedure described in ASTM C597-2009 was followed to measure the dynamic modulus of elasticity using ultrasonic pulse velocity. The percentage reduction in elastic modulus (after 328 freeze-thaw cycles) or the number of freeze-thaw cycles to failure (if the sample broke before completion of 328 cycles) are reported in Table 4.3.

Of the cores tested from US 35 only the sample from the mid-panel of the panel with more deteriorated joints

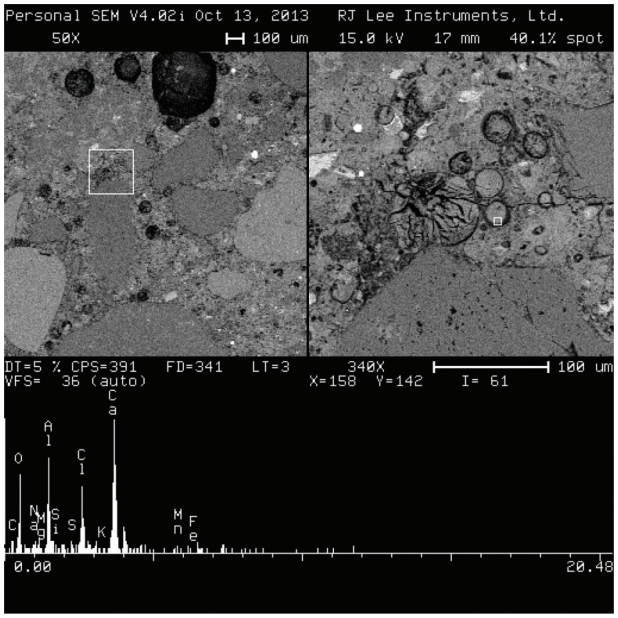


Figure 4.6 Friedel's salt deposit in crack.

failed (US35-D). All other samples tested from US 35 showed a reduction in modulus of elasticity that averaged 35% and ranged from 16.7% to 42.6% (as shown in Table 4.3). Although marginal, the freeze-thaw test results from all US 35 concrete tested are relatively similar, as are the air void parameters. The exception, core D (mid panel core) that failed after exposure to 189 cycles, had cracks that often ran through the aggregates, suggesting that some of the aggregates in the sample tested may have been FT susceptible (even though the concrete was placed in 2003, after the AP specifications for the highest quality, FT durable aggregates were in place).

Although the air void parameters in SR38-R were better than those measured in SR38-25 both sections performed similarly in the FT test. All samples extracted from joint cores from both locations along SR 38 failed before reaching 300 FT cycles, whereas the two samples tested from mid-panel cores did not fail but showed a reduction in modulus of 28% and 42%. These test results suggest that the concrete at the joints in this pavement are more susceptible to FT deterioration.

The freeze-thaw results for both deteriorated and non-deteriorated locations along SR 3 were similar. Even though the measured air void system did not meet any of the criteria for FT durability the mid-panel samples did not fail, but showed approximately 30% reduction in modulus of elasticity. All samples tested from the joint cores failed before completing 100 freeze-thaw cycles. The samples extracted from cores that appeared to be non-deteriorated joints also showed poor FT resistance implying that there is potential for future FT damage.

It may be argued that the FT testing of 1″ thick slices of concrete is a harsh test that may not replicate the field conditions of pavements, however results might be

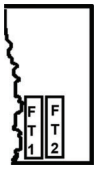


Figure 4.7 Location of freeze-thaw samples for joint cores.

considered indicators of potential response to long-term exposure to severe freezing and thawing.

In summary, the mid-panel cores generally performed better than joint cores. Of the eight joint cores that had two samples tested, five of the FT1 samples closest to the joint face failed sooner, or had a higher reduction in modulus than the samples slightly further from the joint face (FT2). The three cores in which FT1 had similar or better performance than FT2 were all from better performing joints. This observation supports the microscopic analyses and field observations that deterioration is worse, or perhaps starting at the joint face. Examinations and tests also suggest that many of the joints which seem to be performing well in the field might be susceptible to freeze-thaw deterioration in the future based on the air void analysis, SEM examinations and freeze-thaw test results.

4.5 Recommendations for the Greenfield District

Summarizing all the test results for pavements examined in the Greenfield District it is apparent that the newer pavements are showing less deterioration at the joints. For the newest pavement, US 35 built in 2003, there were only minor differences in both surface appearance and test results for panels with "deteriorated" compared to "less deteriorated" joints. Concrete from the joints were fairly similar to mid-panel samples as well. The subbase still had good drainage at all core holes. The air void system was marginal but considering only minor distress and infilling of the air voids were observed rehabilitation may be beneficial. Removing the backer rod and sealing the joint faces with a silane or other water resistance sealer could prevent these joints from deteriorating further and may add several years to the pavement life.

The next youngest pavement, SR3-115 built in 1994 does not show signs of deterioration yet, but test results suggest that this pavement may be prone to joint deterioration. As with US 35, this section of SR3-115 may benefit from removing the backer rod and sealing the joint faces with a silane or other water resistance sealer. This minor amount of rehabilitation could prevent these joints from deteriorating, or greatly reduce the rate of deterioration, and may extend the pavement life.

Both sections of SR 38 examined were built in 1990. However, based on testing, examinations and field observations the section of pavement near the intersection of Reddingdale Dr (SR38-R) is performing better than the section near 25th St (SR38-25). It is likely that removing the backer rod and sealing the joint faces with

a silane or other water resistance sealer could prevent or at least slow the rate of further deterioration at the joints, especially in the pavement near Reddingdale Dr (SR38-R) and in the joints near SR38-25 in the EB lane. Because of the advance state of deterioration in the WB lanes near SR38-25 and the decreased rate of drainage of the base, the benefit of rehab may be more limited.

The oldest pavement examined in the Greenfield District, SR3-113, built prior to 1987, is showing some of the worst deterioration. The aggregate is a quarried carbonate rock and may not be AP quality. Considering the field observations, test results and the slow drainage at the joints, any rehab work is expected to provide only marginal improvements to the overall pavement life.

4.6 Recommendations for Cores Taken from Seymour District and Fishers

Existing cores taken from Seymour District and Town of Fishers are available. Detailed petrographic examination of these cores may provide additional valuable information. Some of the questions that could be investigated include:

1. why are the new concrete patches deteriorating more quickly than the older original concrete;
2. is the microstructure of the older concrete from the 1960s different than more recent concrete and do these differences effect its durability (e.g., if the older concrete was built with cement with less sulfates is there less ettringite in the voids, is there less CH even though fly ash was not use);
3. are there any differences in the microstructure in concrete only exposed to Iceban (Fishers) or never exposed to CaCl (Seymour);
4. can any difference in the concrete at joints be detected where hot pour joint sealer was used compared to silicone sealer with a backer rod;
5. is there a difference in the air void structure in pavements built with MicroAir compared to vinsol resin air entraining agent (AEA);
6. does the microstructure give any clues as to why the concrete at the joints in lanes in one direction are performing differently than the opposing lanes; and
7. did paving during a snow storm affect the concrete microstructure, and perhaps additional questions.

5. CONCRETE CONDUCTIVITY OF LABORATORY MIXTURE

5.1 Introduction

Joint deterioration manifested itself as cracking and spalling of concrete in the vicinity of both longitudinal and transverse joints. Under service conditions, the temperature, pore solution concentration, and degree of saturation of concrete near the joint vary continuously. Electrical conductivity, which also depends on the aforementioned parameters, might offer a means of monitoring changes in those parameters over time and improve our understanding of the cause of deterioration. This work examined how temperature, saturation level, and air content individually affect the electrical conductivity of concrete.

In concrete, conduction occurs predominantly through the pore solution. An increase in the degree of hydration or in the level of drying will increase the tortuosity of the conductive path which then reduces electrical conductivity of the sample. Electrical conductivity of concrete depends on: temperature, moisture content (or degree of saturation) and pore solution concentration (or conductivity).

5.2 General Approach

Concrete cylinders (4″ × 8″) were cast then were cured at 23°C and 100% relative humidity (RH) for at least 28 days. Then 2″ thick disks were cut, vacuum saturated, and exposed to different conditioning as described in the following sections. After conditioning, electrical resistance was measured and electrical conductivity calculated. (For additional details see Appendix D and the TRB 2014 Annual Compendium of Papers, Paper 14-5694, *Concrete Conductivity: Effect of Temperature, Saturation and Air Content* by Panchmatia, P., J. Olek and N. Whiting).

5.3 Electrical Conductivity and Temperature

In concrete, conduction happens predominantly through pore solution (Rajabipour, 2006). Previous work (Crisp, Starrs, McCarter, Rouchotas, & Blewett, 2001; McCarter, 1995; McCarter, Starrs, & Chrisp, 2000) has shown that Arrhenius relationship is applicable to concrete (see Equation 5.1).

$$\sigma = Ae^{-\left[\frac{E_a}{RT}\right]} \tag{5.1}$$

Where, σ is conductivity of concrete (Siemens/meter), A is the nominal conductivity at infinite temperature (Siemens/meter), Ea is activation energy (kJ/mole), R is the universal gas constant (8.314).

Other researchers have verified this relationship for concrete at temperatures ranging from 0°C to 50°C. This research evaluates the applicability of Arrhenius relationship for temperature ranging from -18°C to 23°C.

As shown in Figure 5.1 the plot between natural logarithm of conductivity and the inverse of absolute temperature is a straight line for temperatures ranging from -18°C to 23°C.

5.4 Electrical Conductivity and Saturation Level

Conductivity has a one-to-one relationship with its moisture content. For a porous medium (like hardened cement pastes and concretes) the bulk conductivity (σ_t) can be modeled using Equation 5.2 (Garboczi, 1980; Rajabipour, 2006; Rajabipour & Weiss, 2007).

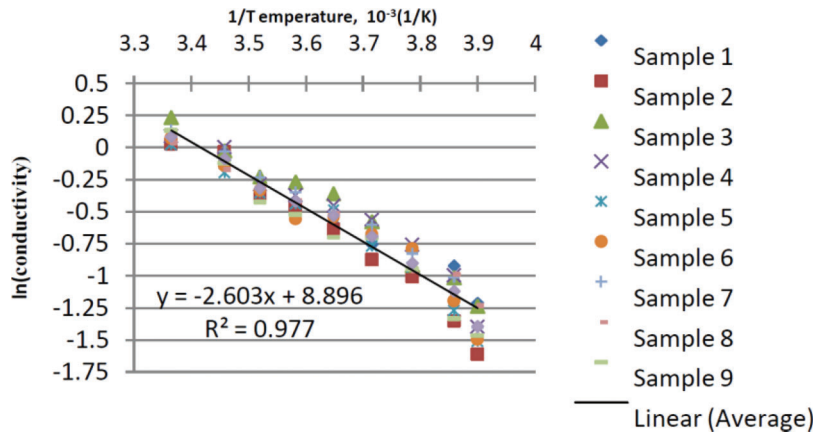


Figure 5.1 Plot of ln (conductivity) vs. 1/temperature for nine specimens.

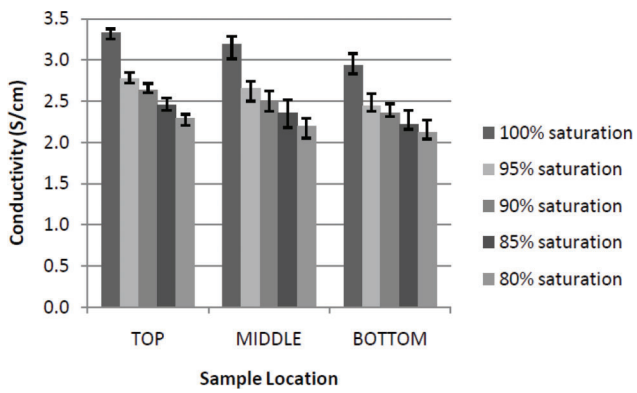


Figure 5.2 Changes in conductivity with changing saturation level of concrete.

$$\sigma_t = \sigma_0 \varphi \beta \tag{5.2}$$

Where σ_0 is pore solution conductivity (S/m), is total liquid filled porosity, and β is the factor representing moisture connectivity. Upon drying, as the saturation level decreases, the conduction path becomes more tortuous and therefore the conductivity is expected to decrease (β decreases).

The concrete mixture and sample preparation are described in detail in Appendix D. The initial resistance was measured immediately after vacuum saturation. Then the samples were allowed to dry to the desired mass to attain saturation levels of 95%, 90%, 85% and 80%. Resistance was measured when each of those saturation levels was attained.

Figure 5.2 shows that the conductivity of concrete decreased with decreasing moisture content. The conductivity decreased by 30% when saturation level is reduced from 100% to 80%. Half of this decrease occurs during the first 5% decrease in saturation level (from 100% to 95%). There was a 7% to 11% decrease in conductivity observed between samples extracted from the top vs. from the bottom of the cylinder at similar saturation levels. This difference in conductivity may be attributed to a slight segregation in the mix.

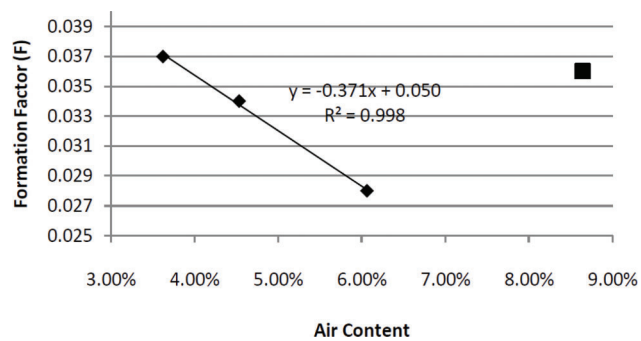


Figure 5.3 Formation factor vs. % air content.

5.5 Electrical Conductivity and Air Content

The formation factor (1/φβ) is defined as the inverse of the product of pore connectivity (β) and total liquid filled porosity (φ). In other words, the formation factor can be interpreted as a measure of the volume of the pores and their connectivity. Therefore, we could use the formation factor to quantify the microstructure of the concrete and hence use that relationship to describe the effect of air content on conductivity of concrete. This study modeled the conductivity of concrete to its air content using formation factor and pore solution conductivity ($\sigma 0$).

Concrete mixtures and sample preparation are described in detail in Appendix D. The formation factor was calculated using Equation 5 in Appendix D. A plot of the results of the measured % air compared to the calculated formation factor for different mixtures are shown in Figure 5.3.

As expected, as the air content increased, the formation factor decreased suggesting that conductivity increased, except for the mixture with the highest air content of 8.6%. The lower conductivity (higher formation factor) for the mixture with 8.6% air suggests that at higher air contents the paste is more difficult to saturate completely. Perhaps at such higher air contents there are more smaller air voids which are more isolated. Empty air voids impede conductivity by increasing the tortuosity of the fluid path in the paste.

To explore this hypothesis the estimated amount of empty or unsaturated air voids in the concrete sample were calculated. The calculations are detailed in Appendix D. To summarize, the mixture with 8.6% air was less saturated at only 64% saturation as opposed to the other mixtures with 4.5% and 6.1% air which were more than 80% saturated.

5.6 Conclusions and Recommendations for Future Work

The following conclusions can be drawn based on this work:

- Concrete conductivity follows Arrhenius relationship for temperatures below freezing and more specifically that range between -18°C and 23°C.
- The concrete conductivity decreased by 30% when saturation level reduced from 100% to 80%.
- Segregation within the concrete mixture can affect the concrete conductivity.
- Concretes with higher air contents are more difficult to saturate compared to concretes with lower air content. Therefore increasing air content might postpone the deterioration of concrete at the joints by freeze-thaw damage.
- When saturated, concrete with higher air content demonstrates higher conductivity.

It is recommended that the results of this preliminary testing be used to design experiments that attempt to quantify the microstructural deterioration, using conductivity, that occurs in concrete when exposed to freezing and thawing in the presence of deicing salt solutions. Comparing conductivity information from laboratory testing with conductivity values obtained while testing samples extracted from deteriorated and non-deteriorated concrete pavements joints may provide further understanding of the reason for premature joint deterioration.

6. PROJECT SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary and Discussion

6.1.1 Introduction

The objectives of this study were (1) to determine what changes have occurred over the past 45 years in pavement materials, designs and construction practices, and current deicing materials and practices that may relate to the deterioration of concrete at some pavement joints and (2) identify the influence of these changes on the durability of Indiana pavement concrete at the joints by examining pavements built with various materials, designs and practices and exposed to various deicers.

To identify the changes and different conditions and how they may have influenced joint durability, historical records specific to each pavement section were needed. In most cases the records of the actual pavement materials used for construction, as built designs, construction records, and deicing materials and practices for specific roads and segments were not available. Since the year built was available for most pavements assumptions were made for each pavement based on the prevailing practices, materials and specifications at the time of construction. Pavements sections were selected for further examination based on field conditions and year built. Additional information was determined and properties verified through field and lab investigation such as joint sealer used, saw cut configuration, fly ash in the mix, air void system, aggregate type, pavement thickness, base condition, etc.

6.1.2 Historical Data

INDOT Specifications: To identify the prevailing specifications and construction practices at the time a pavement sections was constructed the INDOT Standard Specification books from 1952 through 2012 were reviewed. Changes to specifications over time that may relate to joints and concrete durability are summarized in Table 6.1. A specific practice may have occurred prior to these dates if used under a special provision.

Cement: The physical and chemical properties of the cement used in construction of each pavement may have influenced the durability of the concrete, however these records were not available. Published national data and the information from the cement mill certificates on file at INDOT were available and showed that the cement fineness, strength and chemistry have changed over the years. The fineness, measured as Blaine has increased for both the INDOT cements and cements across the country. The amount of C_3S has increased and C_2S decreased over the years. The 1-day

TABLE 6.1
Summary of changes in INDOT specifications.

Year	Relevant Specification
1985	First mention of the use of fly ash (FA) as supplementary cementitious material
1988	First mention of silicone joint sealer being used
1995	Double saw cut at the transverse contraction joints (T-jt) introduced
1995	QA option established for PCCP mixtures and construction which included:
1999	Single saw cut for longitudinal joints (L-jt), tie bar spacing of 3′ and its size adjusted for pavement thickness
2006	Target % air increased to 5.7%–8.9%
2012	Minimum % air before failure raised; tie bar size decreased for thicker slabs

through 28-day strengths all have increased since 1954. The 1-day and 3-day strengths increased most dramatically resulting in increased early-age rate of strength gains. Changes in early strengths likely are in response to the industries' preference for rapid construction practices. The 7-day strengths have been the most consistent values across all cements examined since 1990, which may be in response to the long-standing and common 7-day strength requirement for acceptance. Both the increase in fineness and C_3S contribute to the dramatic increase in 1-day and 3-day strengths since 1954.

The sulfates also have increased since 1954 to avoid set problems that can occur with finer cements and more readily available aluminates (Neville, 1997). Some side effects of these physical and chemical changes in cements to consider include:

- increased development of early heat of hydration that could make concrete more susceptible to problems associated with concreting in high ambient temperatures;
- increased CH in the hardened concrete that has been linked with durability issues, especially in the presence of deicers; and
- increased sulfates in the hardened cements which may, in part, contribute to the formation of ettringite frequently found lining and filling the concrete air void system and reducing the effectiveness of the air voids system for FT protection.

Although additional CH in hardened concrete has been associated with long-term durability concerns, silicates from supplementary cementitious materials (SCM) can combine with CH to form additional CSH (calcium silica hydrates) contributing to long-term strength and durability (Neville, 1997; Sutter et al., 2008). Therefore the use of SCMs are especially valuable with modern cements in reducing the CH and increasing the CSH in the concrete.

Deicing salts can increase the moisture content of concrete leading to a higher level of saturation and susceptibility to freeze-thaw distress (Sutter et al., 2008; Jones et al., 2013). Specific types and quantities of deicers to which each pavement section was exposed could not be identified so the influence of specific deicers and deicing practices on concrete durability at specific joints could not be ascertained. However, the lower amounts of deicers used combined with the exposure to fewer freeze-thaw cycles in the southern INDOT districts of Seymour and Vincennes may be a key factor in why this type of joint deterioration was not identified in the Vincennes District and identified in only one pavement less than 40 years old in the Seymour District.

A database was developed for populating with data and use, however the forms and process are clumsy and incomplete. It is recommended that a more comprehensive and easy to use database be developed and maintained so that the information developed from this and other projects can be combined in searches and easily accessed for many years to come.

6.1.3 Pavement Selection and Field Observations

A total of 45 pavement sections were identified from around the state of Indiana as potential candidates for detailed investigation, many with some level of joint deterioration and some as a non-deteriorated section for comparison. Through an extensive review process that included site visits, district surveys and review of what records were available 11 pavements were selected to represent materials, practices and specification changes from 1960 through 2004. A total of 55 cores were taken at good and deteriorated joints and the corresponding mid-panels. Field observations included rate of drainage of the base, pavement depth, and condition of the joint at and below the surface. Additional testing in the lab on 19 of the cores from the Greenfield District (built from the mid-1980s to 2004) included air void analysis, FT testing and detailed microscopic analysis.

Field observations revealed that the level of joint deterioration was not always obvious from the surface. Some of the joints that appeared to have no or very minor deterioration showed the beginnings of more significant damage just below the joint sealer.

Drainage: There did appear to be a connection between the drainage of the base and the level of deterioration seen at the joint. All pavements were built on a granular base. Over 80% of the pavements had good drainage at the mid-panel but only 40% of these had good drainage at the joints (i.e., the base at the joints of approximately 1/3 of the pavements tested had good drainage). Although pavements with moderate levels of deterioration at the joints had drainage that ranged from good to slow, the joints with some of the worst levels of distress also had the slowest drainage, and joints with no or only minor distress had moderate to good drainage.

6.1.4 Laboratory Analysis of Cores

Air void system analysis: As shown in Table 4.3 the air void systems of the cores examined all had a compromised air void system which led to not meeting all three criteria recommended for FT durable concrete (as summarized in Table 6.2). Over 50% of the cores did not meet even one of the criteria, some from deteriorated joints others from non-deteriorated concrete. The one consistent trend was that for a given panel, the concrete at the mid-panel had a better air void system than at the joint.

Freeze-thaw testing results did not correlate directly with the air void analysis, but generally samples from the mid-panel performed better when exposed to repeated freezing and thawing cycles than the samples taken from the joints. These results are not surprising considering the air void system was compromised by infilling with secondary minerals in the concrete near the joints.

Microscopic/SEM analysis: Concrete from the mid-panel cores and at the joints were examined from the pavement surface (0″) to 5″ depth at magnifications of

TABLE 6.2
Measured air void parameters.

Air Void Parameter	Range of Values	Recommended
Total Air	2.5–5.5%	5.7%–8.9% [INDOT Spec]
Spacing Factor	0.008–0.0261 in	≤0.008 in (Arribas-Colón et al., 2012)
Specific Surface	235–851 in^{-1}	≥600 in^{-1} (Arribas-Colón et al., 2012)

500x or higher. Secondary minerals such as ettringite and Friedel's salt were identified lining and filling voids in all the samples. Secondary minerals form when mineral-rich fluids move into or through the voids. Infilling of voids with ettringite and Friedel's salt indicate repeated ingress of fluid into or through the concrete.

The amount of ettringite increased with depth. Voids lined or infilled with secondary minerals were more common at the joint than at the mid-panel, which could explain, at least in part, the decreased air void parameters measured in concrete from the joints verses mid-panels. The pattern of occurrence for Friedel's salt, a chloride-based secondary mineral, suggests that very little deicers penetrated beyond 2″ in the mid-span of the concrete, however some amount of deicers reached greater depths at joints.

Fly ash was identified in all but one pavement section. Two of the four cores taken at SR38-R did not have fly ash in the concrete and showed more advanced deterioration at the joints than the nearby concrete that did have fly ash.

Other significant observations include the pattern of occurrence of unhydrated cement grains that suggested the bulk of the pavements concrete is adequately cured but the joint faces are not. Calcium hydrate (CH) was observed in the concrete pavement with some of the most severely deteriorated joints examined (SR38-25), supporting the theory that more abundant CH in the paste relates to less durable concrete.

6.1.5 Recommendations for the Greenfield District

Examinations of pavement concrete and subsequent lab testing focused on cores taken from the Greenfield District. Based on the observations in this study some rehabilitation may be cost effective and likely extend the life of some of these pavements. The deterioration of the concrete at the joints could be slowed, if not eliminated, by reducing the concrete's exposure to moisture to the point that reaching critical saturation during freezing events is hindered. A practice that could reduce the potential for concrete to become critically saturated at the joint that is becoming more common is removing the backer rod and sealing the joint faces with a siloxane, silane or other water resistance sealer. Resealing without the backer rod would further reduce the amount of moisture to which the joint is exposed while reducing the influx of water and incompressibles at the joint.

The sections examined in this study that may benefit most from such rehabilitation are:

- US 35 north of Muncie near the intersection with SR 28 and SR 3 (G-US35)
- SR 3 RP 115–117, north of SR 38, New Castle (G-SR3-115),
- SR 38 in New Castle (aka Broad St); the pavement near the intersection with Reddingdale Dr may be a good candidate (G-SR38-R) however the pavement near 25th St may be not because of the present condition of the pavement (G-SR38-25).

6.1.6 Concrete Conductivity of Lab Mixes

Laboratory testing examined the influence of changing temperature, saturation level and air content of hardened concrete on the bulk electrical conductivity test as a step to further development of this test as a tool for determining existing concrete properties. To advance this technique for use in field concrete additional work is needed to correlate changes with microstructural deterioration and changes in concrete that may occur when exposed to freezing and thawing in the presence of deicing salt solutions.

6.2 Conclusions

Several changes in concrete materials and pavement design and construction practices over the past 50 years were identified that could influence the durability of the concrete at the joints. Based on the information collected and pavements examined, the following are considered probable contributors to the durability of concrete at the joints in Indiana.

The identification of higher CH in the concrete from the pavement with severe deterioration at the joints supports the theory that increased amounts of CH in the hardened concrete can lead to durability issues, especially in the presence of deicers. Fly ash was identified in the concrete of the pavement with severe joint distress that should have helped convert some of the CH to CSH. Without knowledge of the chemistry of the cement or the fly ash used in that pavement we cannot know if the cement contained an unusually high C_3S:C_2S ratio that would have contributed to a high CH content and if the amount and chemistry of the fly ash would have been adequate to convert the available CH.

Pavements both with and without deterioration at the joints contained fly ash. One section of one pavement examined did not have fly ash, and it did

have more deterioration at the joints than the sections of this pavement that contained fly ash. No other differences were detected other than the presence and absence of fly ash. Although the presence of fly ash did not prevent the deterioration at the joint it has reduced the severity of the deterioration.

The increased sulfates in the modern cements likely contribute, at least in part, to the formation of ettringite found lining and filling the concrete air void system. This phenomenon is reducing the effective air void system, especially at the joints. If the original air void system just marginally met the criteria for FT durable concrete than any amount of lining or filling by secondary minerals could lead to less durable concrete. The higher occurrence of secondary minerals in the concrete near the joints indicate that more moisture was moving into or through the concrete at the joints then through the bulk of the concrete pavement.

An adequate air void system is critical to freeze-thaw durability of concrete. Recent changes to INDOT's specifications for acceptable air content measured at the construction site is expected to improve the quality of the air void system in recently placed and future concrete pavements.

The ability of the base to drain water from the joints is linked to the durability of the concrete at the joint. The granular base in place at most of pavement sites cored was considered good and able to dissipate the drilling water during or within a minute or two after coring. However this ability to drain water was reduced significantly at two-thirds of the joints where this property was measured. The reason for this decrease was not determined, but the joints with the most severe deterioration corresponded to the worst drainage.

Unhydrated cement grains were often common near the joint face indicating that the concrete in the top 2″ of the joint face is not always fully curing.

The two southern districts of Seymour and Vincennes exhibit less concrete joint durability problems than the more northern Districts that historically have been exposed to higher amounts of deicers, and harsher, longer winter environments. This supports the theory that prolonged exposure to deicers and FT cycles, and the rate and duration of freezing likely influences the potential durability of the concrete especially at the pavement joints.

Several variables were identified that influence the durability of the concrete at the joints and there may be other variables that were beyond the scope or capacity of this study (such as original paste quality, pate density, w/cm ratio). In summary, this study identified that one or more of the following variables likely influenced the durability of the concrete at the joints examined: the drainability of the base at the joints, original air void system, reduced air void parameters due to lining and infilling of the air voids with secondary minerals, compromised hydration of the concrete at the joint face, increased moisture at the joint.

6.3 Recommendations

6.3.1 Practices to Consider for Optimal Durability

Fly ash or other SCMs that provide additional silica are recommended as part of the cementitious mixture in order to help convert CH into CSH, which is especially critical if the cement has a higher C_3S:C_2S ratio, as is common in modern cements. Optimizing the amount of fly ash in a particular mixture has been shown to improve the paste density and is expected to reduce moisture migration (Rudy & Olek, 2012). Also, many modern cements are more susceptible to higher heats of hydration. SCMs that reduce the heat of hydration are especially valuable when concrete is placed during high ambient temperatures.

Rehabilitation: If the concrete at the pavement joint is beginning to deteriorate and the concrete is believed to contain FT durable aggregate and an adequate air void system, then rehabilitation that includes removing the backer rod and sealing the joint face with a silane or other penetrating waterproofing sealer should be considered. (See recommendations for the Greenfield District for more details.)

Sealing of joints without a backer rod may reduce the amount of moisture held at the joint face that contributes to the concrete reaching the critical saturation level that renders it susceptible to FT damage. Treating the joint face with a silane or other penetrating waterproof sealer soon after sawing may improve the curing of the concrete at the joint face making the concrete more durable, and may reduce the potential of the concrete to become critically saturated throughout the life of the joint (as suggested by previous research (Jones et al., 2011)).

Air void system is commonly reduced in older pavements. Adopting practices that give an excellent original air void system is valuable, encouraging spacing factors and specific surface that are much better than marginal. These parameters are critical to long-term durability of concrete exposed to a harsh FT environment and deicers. A balance between optimal air for long-term durability and meeting early strengths requirements needs to be considered.

Drainage: Ensuring the hydraulic conductivity of the base, especially at the joints is adequate and remains good throughout the life of the pavement.

6.3.2 Additional Research

This study focused INDOT's and industry's attention on some common issues related to durability of pavement concrete at the joints. Some of the findings in this study reinforce and expand on findings from the previous study (Arribas-Colón et al., 2012) and included: (1) moisture present in the joints plays a critical role physical deterioration of concrete during the freezing and thawing cycles, (2) the effect of the increased moisture levels on the integrity of the air-void system, (3) the heightened potential for the chemical

damage of the microstructure in the presence of deicing chemicals and (4) the beneficial effects of using supplementary cementitious materials (mostly fly ash) in reducing the prospect of both, the physical and the chemical damages. These observations led to additional research efforts focused on more in-depth understanding of the effects of some of these variables on pavement concrete durability and devising ways of addressing these problems in the future. Some examples of these continued research efforts include SPR-3864, *Performance of Deicing Salts and Deicing Salt Cocktails* (Suraneni et al., 2016), with the focus on examining how the composition of different deicing salts influences the potential for initiation of deleterious reactions that can cause damage to the pavements; SPR-3808, *Synthesis: Accelerating the Implementation of Research Findings to Reduce the Potential Concrete Pavement Joint Deterioration,* the objective of which is to comprehensively review and synthesize all suggested causes and phenomena associated with joint deterioration for purposes of developing a manageable solution to prevent and/or to mitigate this problem in the future; and SPR-3523, *Evaluation of Sealers and Waterproofers for Extending the Life Cycle of Concrete* (Wiese et al., 2015), which focuses on the use of sealers and waterproofers for concrete paving materials with specific intention of improving joint behavior.

Further work can increase our understanding of the dynamic processes that contribute to problems with concrete durability at pavement joints and may lead to an understanding of how to avoid future related problems. To that end the following additional work is recommended.

Air void system parameters that are considered adequate for freeze-thaw may need to be reestablished considering the changes and advances in cementitious mixture designs, air entraining agents, and other admixtures, as well as anti-icing/deicing products and practices now commonly used in Indiana. Both durable and non-durable concrete did not meet existing recommendations. A better understanding is needed as to how mixture designs, mixing, placement practices, materials and other variables influence the in-place total air, spacing factor and specific surface of the air void system. This may lead to recognizing variables that can be measured and controlled even if the air void parameters other than total air cannot be directly measured in the field.

Existing cores taken from the Seymour District and Town of Fishers are available for additional work. Detailed petrographic examination of these cores may provide additional valuable information and relevance of joint deterioration to concrete age, joint sealer, deicers and more. See section 4.6 for more detailed suggestions on what questions could be investigated because of the strategic selection of these cores.

Drainage of the base is critical to long-lasting durable pavements. Understanding why/how the hydraulic conductivity of the base at the joints was reduced may help to avoid or prevent this in the future. Additional work may be needed to look at the influence of design, construction, piping of fines up from the subgrade, fines moving down through the joints into the base, the breakdown of base materials or other mechanisms that could be reducing the hydraulic conductivity of the base under concrete pavements at the joints.

Database development: Retain as much detailed information as possible on materials used, construction practices, mix designs, pavement designs, ambient conditions during paving, original air void parameters, the physical and chemical properties of cements used, type and quantity of deicer exposures and more, for each specific pavement section in a form such that the data will be available for decades. An easily accessed electronic database could provide a good platform for retaining and tracking information, and allow for useful queries. INDOT's Site Manager database has some of these capabilities. Building a comprehensive database that included test results and analyses from studies could enable more detailed and accurate correlations that could be more thoroughly analyzed between any of these variables and any concern that may arise related to the durability of specific concrete pavements or other durability concerns.

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APPENDIX A: HISTORICAL DATA

A.1 PUBLISHED DATA

Table A.1 and Table A.2 show data published for average cement strengths from 1953–54 and 1994. Table A.3 shows historical published data of the average oxide analysis of cements.

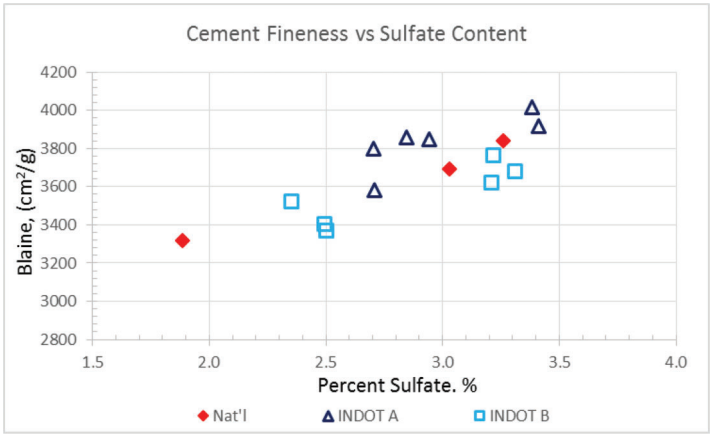

Figure A.1 Comparing cement fineness to percent sulfates in cements.

TABLE A.1
Strength values in psi for mortars prepared using cements produced in 1953–54 (1).

	1 days	3 days	7 days	28 days
Mean	915	2148	3331	5043
STD	221	427	550	715
n	97	97	97	97
Max	1720	3310	4900	6790
Min	120	550	2200	3100

TABLE A.2
Strength values in psi for mortars prepared using cements produced in 1994 (2).

	1 days	3 days	7 days	28 days
Mean	2149	3612	4663	5959
STD	436	339	342	506
n	46	69	68	56
Max	3130	4480	5450	7450
Min	1430	2900	3720	4507

TABLE A.3
Average cement chemistry from published data.

	Cement Chemical Composition, %						Loss on	Potential Phase Composition			
Year	**SiO_2**	**Al_2O_3**	**Fe_2O_3**	**CaO**	**MgO**	**SO_3**	**Ignition**	**C_3S**	**C_2S**	**C_3A**	**C_4AF**
1953–54	21.39	5.68	2.73	64.04	2.31	1.88		50.78	23.11	10.41	8.29
1994	20.55	5.41	2.59	63.91	2.09	3.03	1.37	53.7	18.4	10.0	7.9
2004	20.17	5.07	2.66	63.23	2.51	3.26	1.52	56.9	14.8	8.9	8.2

A.2 INDOT FILES

The following is a summary of cement data collected from Mill Certificates on file at INDOT (Table A.4) and the deicer use data from INDOT districts (Table A.5).

TABLE A.4
Average cement chemistry for INDOT cements A and B.

	Cement Chemical Composition, %						Potential Phase Composition			
INDOT Cement A	**SiO_2**	**Al_2O_3**	**Fe_2O_3**	**CaO**	**MgO**	**SO_3**	**C_3S**	**C_2S**	**C_3A**	**C_4AF**
1990	21.25	5.33	2.99	64.07	1.92	2.94	51.00	22.70	9.04	9.14
1994	21.47	4.70	3.31	64.37	1.97	2.71	54.42	20.39	6.86	10.05
1998	21.37	4.83	3.32	64.58	1.63	2.71	55.42	19.50	7.18	10.09
2002	21.06	4.63	3.45	63.66	1.74	2.85	54.89	18.89	6.44	10.50
2006	20.23	4.93	3.28	62.58	1.93	3.41	53.75	17.38	7.38	10.00
2010	20.29	4.75	3.23	62.79	2.28	3.38	55.33	16.58	7.00	10.00
INDOT Cement B										
1990	21.00	5.46	2.13	65.45	1.413	2.50	60.86	14.32	10.71	6.55
1994	21.28	5.35	2.27	65.43	1.219	2.50	58.39	17.04	10.43	6.73
1998	20.97	5.35	2.38	65.29	1.801	2.35	60.53	14.61	10.30	7.18
2002	20.79	5.47	2.48	64.61	1.159	3.21	55.70	17.70	10.10	7.40
2006	20.50	5.47	2.45	64.04	1.350	3.31	55.13	17.13	10.25	7.38
2010	19.01	5.59	2.98	63.25	3.271	3.22	62.06	7.68	9.77	9.07

TABLE A.5
Deicer usage for INDOT districts for the past 3 years.

Comments	District	Subdistrict	Subdistrict Code	Salt (STN)	Brine (GAL)	$CaCl_2$ (GAL)	$MgCl_2$ (GAL)	$CaCl_2$ (LB)	IceBan (GAL)
Control 1	Symr	Bloomington	5300	39898.35	1196615.03	0.00	110.00	0.00	0.00
Control 2	Symr	Aurora	5200	29935.13	139285.00	0.00	0.00	0.00	0.00
Control 3	Symr	Gary	5100	24401.07	123291.00	0.00	0.00	0.00	0.00
Control 4	Symr	Madison	6100	23961.52	65033.00	0.00	0.00	0.00	0.00
Effect of $CaCl_2$	Ft. Wn.	Warsaw	2200	45128.78	12202.00	174185.30	0.00	0.00	0.00
Effect of $CaCl_2$	Ft. Wn.	Fort Wayne	2400	40659.67	0.00	31144.50	0.00	3650.	0.00
Effect of $CaCl_2$	Ft. Wn.	Angola	2500	44922.28	27646.50	128016.27	0.00	850.	0.00
Effect of $CaCl_2$	Ft. Wn.	Wabash	2600	50977.50	126545.00	103801.50	0.00	1055.	0.00
Effect of $CaCl_2$	Vinc	Paoli	6500	26368.61	84150.00	41732.75	0.00	5800.	0.00
Effect of $MgCl_2$	Craw	Cloverdale	1500	40245.47	201368.50	0.00	6555.	0.00	0.00
Effect of $MgCl_2$	Symr	Columbus	5400	32469.68	362490.00	0.00	3875.	0.00	0.00
Effect of $MgCl_2$	Symr	Fall City	5500	25146.96	471517.00	0.00	10190.	0.00	0.00
Effect of $MgCl_2$ and $CaCl_2$	Craw	Crawfordsville	1200	46039.77	111222.00	11292.63	33626.28	0.00	0.00
Effect of $MgCl_2$ and $CaCl_2$	Craw	Fowler	1300	42806.20	112031.00	5310.00	16300.00	0.00	0.00
Effect of $MgCl_2$ and $CaCl_2$	Craw	Frankfort	1400	37278.64	139308.40	9382.40	28498.00	0.00	0.00
Effect of ICE-BAN	LaPorte	LaPorte	4200	19984.96	755068.80	1150.00	0.00	0.00	33403.50
Effect of ICE-BAN	LaPorte	Plymouth	4400	38026.50	286294.50	0.00	0.00	0.00	12551.50

APPENDIX B: DATABASE DEVELOPMENT

Figure B.1 and Figure B.2 are examples of the information possible to record and the resulting forms developed under the database work.

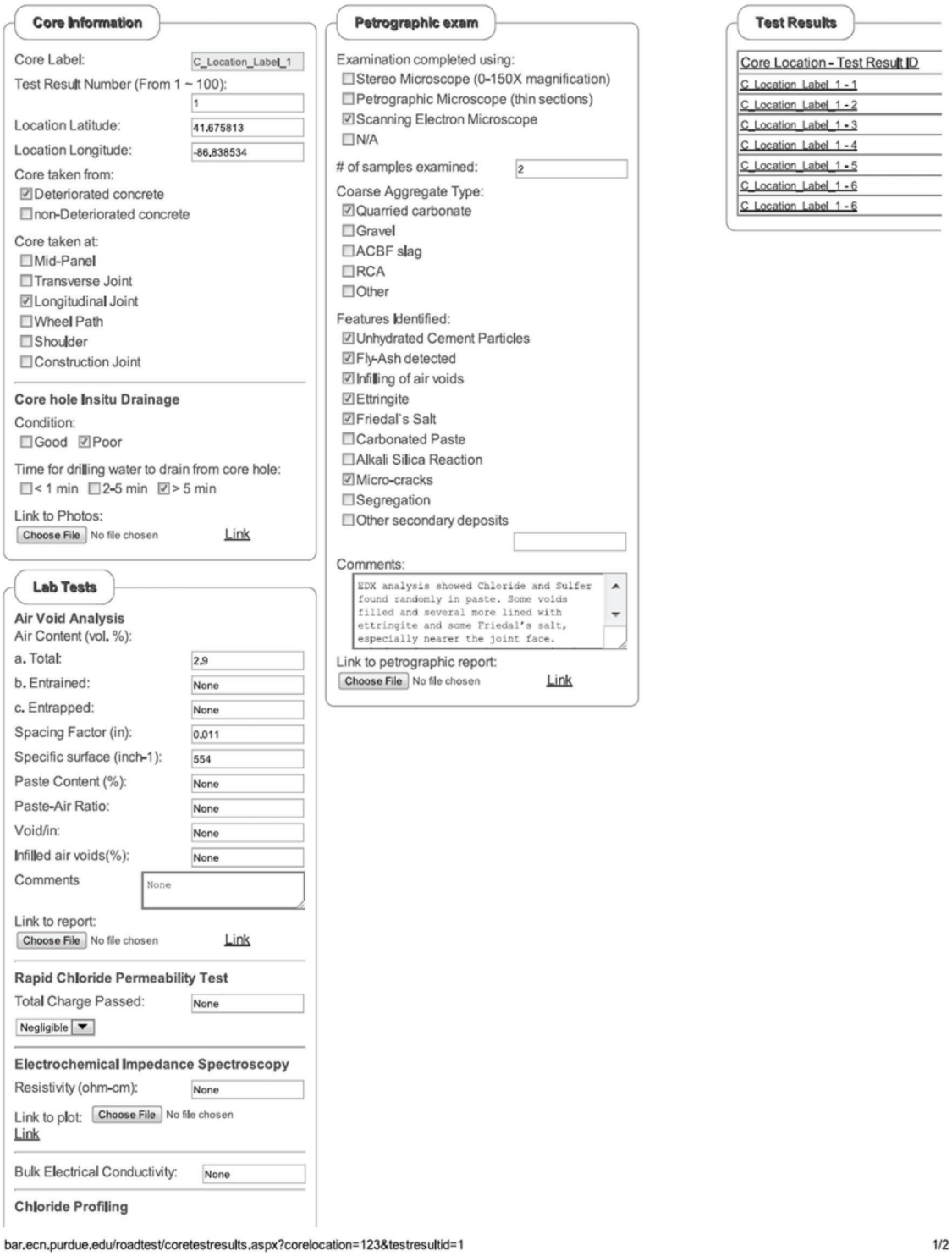

Figure B.1 Form for test results and core location information.

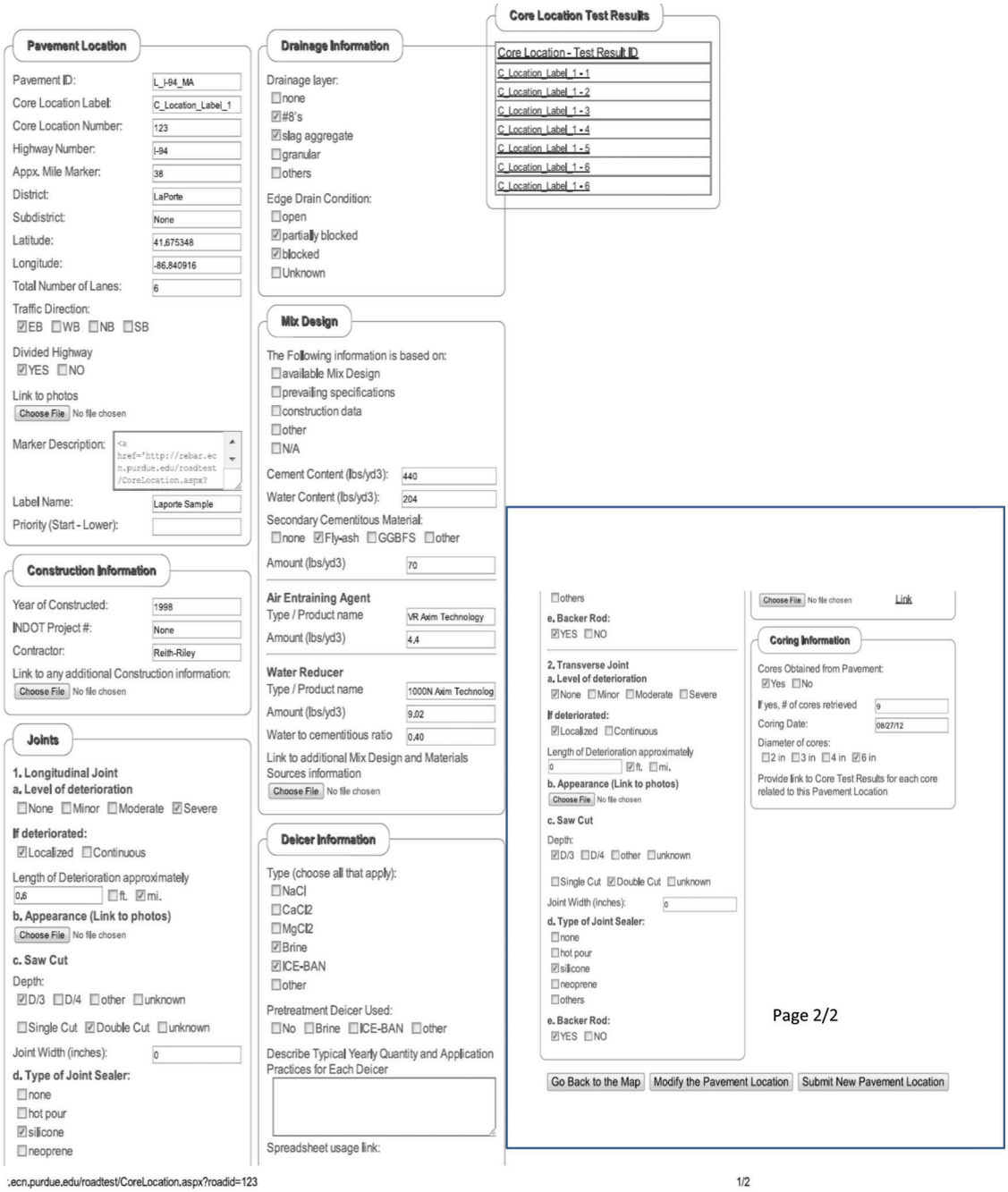

Figure B.2 Example of form for pavement and deicer information.

APPENDIX C: FIELD SITE VISITS AND CORING

C.1 OVERVIEW

Core ID

A core labeling system was developed to be able to quickly distinguish where each core came from and to avoid confusion during lab testing. Each core was labeled in the field upon extraction and each sample cut from that core in the lab was labeled with the unique label. The labeling system is summarized in Figure C.1.

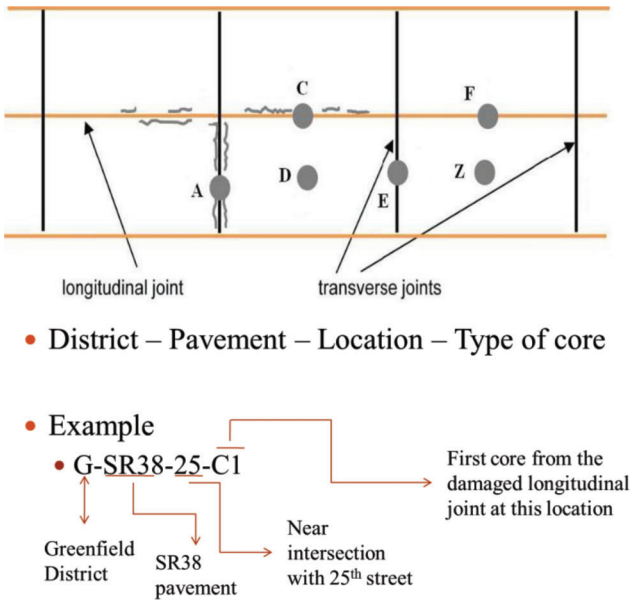

Figure C.1 Core identifier.

Benchmark for Severity of Joint Deterioration

While selecting locations for coring different degrees of deterioration were observed. This section attempts to standardize the nomenclature used in this document to describe the different degrees of deterioration observed in INDOT concrete pavements.

The degree of joint deterioration are classified into five groups of severity: (a) sound joints with no deterioration, (b) minor deterioration, (c) moderate deterioration, (d) moderately severe deterioration and (e) severe deterioration. Photographs showing examples of each of the above mentioned degree of deterioration are shown in Figures C.2–C.6.

C.2 GREENFIELD DISTRICT

Based on information provided by the SAC and Greenfield District five pavements were identified as potential candidates with premature joint deterioration and investigated further.

Location A

The first site was SR 38 New Castle, RP 92.5–94, also known as Broad St between N. 29th St and Reddingdale

Figure C.3 Examples of minor joint deterioration.

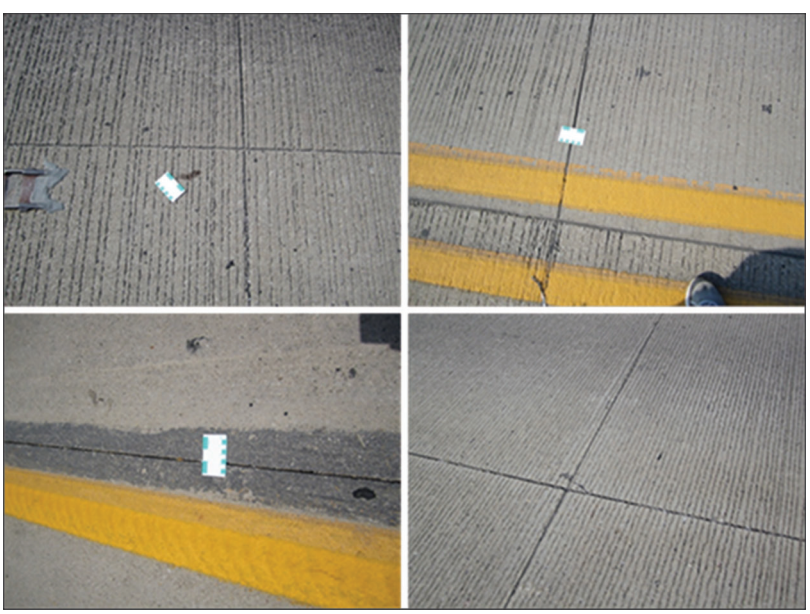

Figure C.2 Examples of sound joints with no deterioration.

Figure C.4 Example of moderate degree joint deterioration.

Figure C.5 Example of moderately severe joint deterioration.

Dr, and labeled A on the site map in Figure C.7. The concrete pavement is primarily in-town curb and gutter construction with storm drains. The coarse aggregate is gravel and pop-outs are very common. Deterioration is evident at both the longitudinal and transverse joints. Transverse joint spacing varied from 15′ to 16′ and 22′. Transverse cracks were common in 22′ long panels, with some of these cracks retrofitted with dowel bars. Drainage at many joints appeared poor as recent rain water sat in several of the transverse and longitudinal joints. Deterioration created widening of joints by as much as 6″.

In the SR 38 pavement from 25th St and east the transverse joints in the eastbound (EB) direction appeared to be less damaged than those in westbound (WB) direction (as shown in Figure C.8). Although pop-outs are common throughout, west of 25th St to Reddingdale Dr the joints show less deterioration (as seen in Figure C.9).

A total of 12 cores were obtained from SR 38. Six cores were obtained from panels just east of the intersection with 25th St (SR 38-25). The latitude and longitude of this location are 39.929817 and -85.344543 respectively. Six cores were obtained from pavement

panels near the intersection with Reddingdale Dr, latitude 39.931264 and longitude -85.360969. There was a date stamp on this location suggesting the pavement was constructed on 6/14/90.

The pavement was built on a granular aggregate base and the drainage characterized by observing how quickly the water used for coring drained from the core hole. As shown in Table C.1, drainage at the pavement near 25th St (SR38-25) that showed more joint deteriorated ranged from very good to poor, and in the pavement near Reddingdale Dr (SR38-R) that had less deterioration at the joints and the drainage ranged from very good to moderate.

The pavement along SR 38 is nominally 10.5″ thick, and the concrete coarse aggregate is a gravel with a nominal top size of 1″.

Figure C.6 Examples of severe joint deterioration.

Location B

Location B is SR 3 and located on the west end of New Castle between RP 112 and 117 with RP 112–115 south of the intersection with SR 38, and RP 115–117 north of this intersection. The concrete pavement between RP 113 and 115 south of SR 38 is 5 lanes wide, pop-outs are common and most of the longitudinal joints in south bound direction show moderately severe deterioration (as shown in Figure C.10). No

Figure C.8 SR 38 near 25th St, New Castle, IN (SR38-25).

Figure C.7 Greenfield District site visit locations with coring locations highlighted (A, B and E).

significant deterioration at the joints was observed along the transverse joints. In contrast, the concrete pavement north of SR 3, between RP 115 and 117 is 4 lanes wide and the joints appeared in good condition (as shown in Figure C.11).

A total of six cores were obtained from SR 3, three from pavement south of SR 38 that showed deteriorated joints (SR3-113), and three north of SR 38 that showed little to no deterioration (SR3-115). The three cores that represented panels with deteriorated joints were taken from pavement panels between RP 112 and 113 in the south bound (SB) lane (outside driving lane), next to mail box 3725. Upon close inspection the transverse joints appeared in good condition because most of them have been patched. After careful consideration a deteriorated transverse joint from the original concrete was located and core. The latitude and longitude of the coring location are 39.896163 and -85.385457 respectively. Three cores were obtained from location near mile marker 115 that represented

Figure C.10 Example of pavement along SR 3 between RP 112 and 115, south of SR 38.

Figure C.9 SR 38 near Reddingdale Dr (SR38-R).

Figure C.11 Example of SR 3 between RP 115 and 117, north of SR 38.

TABLE C.1
Drainage of base at core holes during coring.

Core Label	Drainage of Base (25 = 25th St; R = Reddington St)
G-SR38-25-A1	
G-SR38-25-C1	Very good, no water in the core hole
G-SR38-25-C2	Good
G-SR38-25-D1	Initially good then slowed to 0.25″ drop in 12 minutes
G-SR38-25-E1	Slow
G-SR38-25-E2	Moderate
G-SR38-R-A1	Poor, may not reflect base drainability because core retrieved was not full depth.
G-SR38-R-C1	Good, water drained in less than 5 minutes
G-SR38-R-D1	Good, water drained in 2 minutes
G-SR38-R-E1	Moderate, water level in the core hole changed from 3″ to 2″ in 5 minutes
G-SR38-R-F1	Very good, no water left in core hole
G-SR38-R-Z1	Moderate, water level in the core hole changed from 1.5″ to 0.75″ in 5 minutes

good joints. The latitude and longitude of this coring location are 39.936273 and -85.382966 respectively. As shown in Table C.2 the drainage was good at all the mid-panel core holes but slower at the longitudinal joints in both the deteriorated and non-deteriorated sections.

In the deteriorated pavement section concrete was approximately 9″ thick, the coarse aggregate was a quarried carbonate aggregate nominally 1.5″ top size and the base was a coarse sand. Cracks were visible at the joint face just below the saw cut. In the non-deteriorated pavement concrete was approximately 10.5″ thick, the coarse aggregate was a gravel nominally 1.5″ top size and the base was a #8 crushed stone.

Location C

Location C was along West Memorial Dr (aka 12th St) near US 35 and SR 3 in Muncie, IN. This concrete pavement is in very poor condition with serious deterioration at the transverse joints (as shown in Figure C.12). This pavement was not cored because the distress is likely D-cracking and not relevant to this project.

Figure C.12 Location C, with joint distress that is likely D-cracking.

Location D

Location D was situated on US 35 at the intersection of Eaton St near STA 170. The panals are long with approximately 35′ between transverse joints. Transverse mid-panel cracks are common. Most of the deterioration appears to be associated with the mid-panal cracks and along a longitudinal joint associated with a ramp that appears to be added after the original pavement was placed.

Location E

Location E was north of Muncie, IN at the intersection of US 35, SR 28 and SR 3. The condition of the concrete pavement immediately north, east and west of this intersection appeared very similar. Both longitudinal and transverse joints show minor deterioration or raveling (as shown in Figure C.13).

A total of seven cores were taken along US 35, Muncie, Indiana, west of the intersection with SR 3 (US35-A1 through Z1 as shown in Table C.3). Although most panels showed some minor amount of deterioration at the joints, three cores were obtained from a good panel showing very minor deterioration and four from a panel showing more deterioration. The latitude and longitude of the "good" panel were

Figure C.13 US 35, near the intersection of SR 28 and SR 3, north of Muncie, IN.

TABLE C.2
SR 3 drainage and other field information collected during coring.

Core Label	Drainage	Saw Cut	Comments
G-SR3-113-AE1	Slow	2.5″	Clean edges and no deterioration at core hole; silicone joint gone
G-SR3-113-C1			Quarried CA aggregate; coarse sand base
G-SR3-113-D1	Good, no water left in core hole		Quarried CA aggregate; coarse sand base
G-SR3-115-E1	Good, no water left in core hole	2.5″	Gravel CA; #8 crushed stone as drainage layer; Backer rod with silicone sealer; 3/8″ joint opening
G-SR3-115-F1	Slow, water in core hole after 8 min	2.5″	Backer rod & sealer in good condition; 3/8″ joint opening
G-SR3-115-Z1	Good, no water left in core hole		

TABLE C.3
US 35 drainage and other field information collected during coring.

Core ID	Drainage	Comments
G-US35-A1	Poor	Dowel bar interfered with obtaining a full depth core
G-US35-A2	Poor	Dowel bar interfered with obtaining a full depth core.
G-US35-C1	Very good	Light tapping required to remove core from drill bit
G-US35-D1	Very good	No water left in core hole
G-US35-E1		Core stuck in drill bit; hammered hard to remove
G-US35-F1	Very good	Did not core full depth of pavement,
G-US35-Z1	Very good	Drilling water drained before core removed

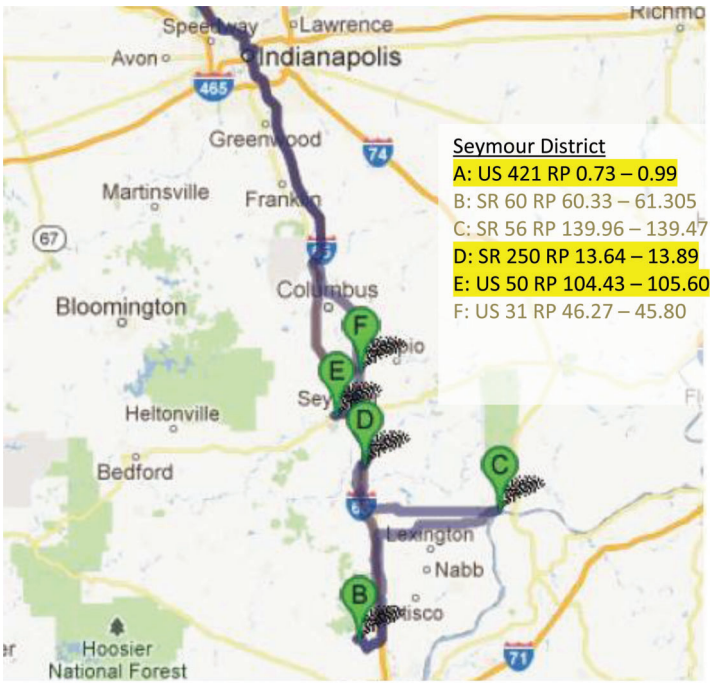

Figure C.14 Seymour District site visit locations with coring locations highlighted.

40.276967 and -85.370156 and that for the deteriorated panel were 40.276943 and -85.370636.

Table C.3 lists the various cores obtained, information on drainage of the drilling water and other observations made during coring. The concrete in the "good" panel had very small sliver spalls along the transverse joint, and short intervals along the longitudinal joint showed some deterioration. From the deteriorated transverse joint (cores A1 and A2) only two half cores were obtained (approximately 5″ deep) because of encountering dowel bars (despite the use of a dowel-bar locator). Because the full pavement depth was not cored at this location the poor drainage at these core holes reflects the drainablity of the joint but may not accurately reflect the drainability of the base. The base was a quarried coarse aggregate mixed with a natural sand and drained quickly in nearly all locations.

The concrete at this location is approximate 12.2″ thick, the CA is a quarried carbonate with nominally 1.5″ top size, built on a good drainable base made up of quarried CA and sand. Of the areas cored the transverse joint saw cut was 2.9″ to 3.5″ deep (~D/4), and the longitudinal saw cut was 5″ deep (~D/2.5).

Interstate Locations (I-65 and I-70)

Because of the high volume of traffic, safety concerns and restrictions of setting up traffic control on these major arteries to collect detailed observations and then core, this study focused on non-interstate locations whenever possible. Video logs were reviewed for the interstate locations identified.

Based on the video log information for I-65 Greenfield District between RP 112.4 and 110.8 collected in 2008, many of the longitudinal joints showed moderately severe deterioration whereas transverse joints showed minor to moderate deterioration. By 2010 both the longitudinal and transverse joints showed

moderately severe and less localized deterioration. Overall, between 2008–2010 the rate of deterioration was fast for this stretch of interstate.

Based on the video logs I-70 in the Greenfield District between RP 145 and 150 both longitudinal and transverse joints showed minor to moderate deterioration which was more localized compared to 2010 where both the longitudinal and transverse joint showed more moderate level of deterioration throughout the section.

Figure C.15 Example of minor (left) and moderate (right) level of joint deterioration common along US 421.

Figure C.16 Fine map cracking seen on the surface of the untined pavement.

C.3 SEYMOUR DISTRICT

Based on information provided by the SAC and Seymour District six pavements were identified as potential candidates with premature joint deterioration and investigated further (locations shown in Figure C.14).

Location A

Location A was US 421 near Madison, Indiana from the intersection with SR 56 north to a small asphalt paved bridge, RP 0.73–0.99, latitude 38.740215 and longitude -85.377321. This section of US 421 is a 4-lane divided highway with curbs, a parking lane both directions and occasional turn lanes. Although INDOT records suggest this pavement was built in 1969 with the latest contract (for repairs) issued in 1996 most of the pavement surface is tined, a practice that began much later than 1969. The turn lanes were not tined and a faint map cracking was apparent on the surface. It is possible that these untined pavement sections are the original 1969 pavement and current driving lanes that are tined were added at a later date. The coarse aggregate is a quarried carbonate aggregate.

Both the transverse and longitudinal joints in the tined concrete on US 421 show minor to moderate deterioration. Much of the deterioration was in the form of small, long sliver spalls along the joint (as shown in Figure C.15). NaCl (solid and brine) was reportedly the only deicer used from 2008–2010.

Seven cores were taken from this section of US 421, three from a panel with moderate joint deterioration, three from a panel that showed very little joint deterioration and one from a panel that was not tined (core V). The pavement surface at core V had fine map-cracking (as shown in Figure C.16) that appeared to be orientated sub-parallel to the longitudinal joint, possibly suggesting a vibrator trail.

Location B

Location B was in Clark County, Indiana on SR 60 RP 60.33 to 61.305, approximately 1 mile west of I-65 and north of Clarksville, Indiana. Clark County is in Fall City Subdistrict which, besides NaCl, logged the use of 10,190 gal of MgCl deicers from 2008–2010.

Figure C.17 Example of sound concrete joints along SR 60 near Clarksville.

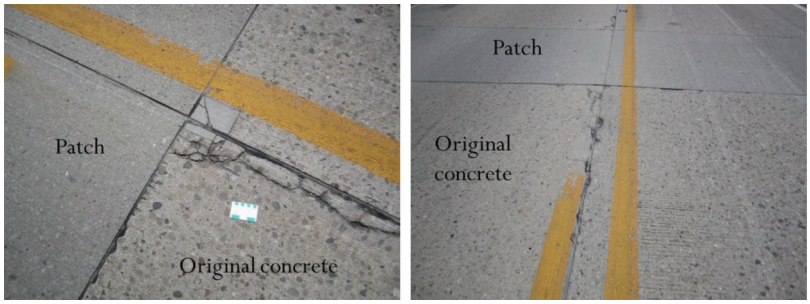

Figure C.18 Example of original concrete and patches along SR 56.

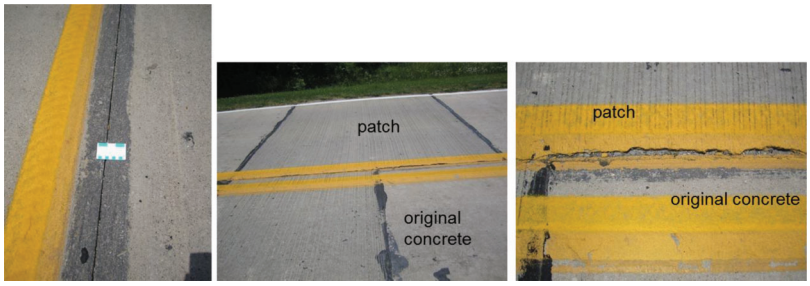

Figure C.19 Examples of SR 250 near Uniontown east of I-65.

Figure C.20 Example of good (left) and moderately deteriorated (right) longitudinal joints in US 50.

This section of SR 60 was built in 2002. The concrete pavement is a 4-lane undivided highway with concrete shoulders. The joints are approximately 3/8″ wide, sealed with silicone joint sealer and appear to be sound and in excellent condition (as seen in Figure C.17).

Location C

Location C was along SR 56 near Madison, IN. This section of SR 56 is a 2-lane highway (aka Clifty Hollow Road) that runs in front of Clifty Creek power plant just north of the Ohio River. The coarse aggregate is gravel and the Madison Subdistrict records indicate that NaCl is the only deicer used (2008–2010).

The pavement is heavily patched with original joints completely replaced in most panels. The existing joints exhibit minor to moderate deterioration at isolated locations in both longitudinal and transverse joints (shown in Figure C.18). The only data available on the age of the pavement is that it is "an older pavement." Considering its potential age and the extensive concrete patching it likely was built prior to INDOT's AP specification and possibly D-cracked. This pavement was not cored.

Location D

Location D was along SR 250 just west of I-65 near Uniontown, Indiana, RP 13.64 and 13.89. It is a jointed reinforce concrete pavement (JRCP), 2-lane rural highway built in 1960, and most recent repairs completed in 2008.

Many of the concrete joints along this section of SR 250 appear to be in very good condition (as shown in Figure C.19, left). Both hot pour and silicone sealant were used at the joints. The panels are approximately 25′ to 28′ long and both longitudinal and transverse mid panel cracks are common.

Full-width concrete patches have replaced a few of the transverse joints (as shown in Figure C.19, center). A few longitudinal and transverse joints show minor to moderate deteriorated, more so in the patches than in the original concrete (as seen in Figure C.19, right). The occurrence of deteriorated and replaced transverse joints increases towards I-65.

Figure C.21 Example of good sound joint (left), slight spalling at joint (center) and patched joint (right).

Six cores were taken from this pavement section, three from a panel showing joint deterioration and three from a panel showing very little deterioration.

Location E

Location E was along US 50 in the town of Seymour, Indiana, near East Tipton St (across from Arby's restaurant). This is a busy 4-lane divided highway pavement built in 1960 with additional work completed in 2007. The primary deicer used from 2008 to 2010 in this area is NaCl, both solid and as a brine, with possibly some very small amount of MgCl.

Although the transverse joints appeared sound and in good condition full panel-width patches are common. Longitudinal joints showed moderate levels of deterioration at isolated locations (as shown in Figure C.20). Joint deterioration is common in both the original concrete and in some patches. Much of the pavement surface has been diamond ground. A quarried carbonate course aggregate was used in both the original concrete and the patches. Some faint map cracking is noticeable. The joints are wider suggesting the joint was made with a double saw cut.

Six cores were taken from this section of pavement: two from a panel of original concrete that showed some deterioration at the joint, two from a patch that had moderately deteriorated joints and two from a panel of the original concrete that had good sound joints.

Location F

Location F was along US 31, a 2-lane undivided highway south of US 50 near the town of Seymour, Indiana, at latitude 38.916470 and longitude -85.832244. Information on the year built was not available but because of the tining and good condition it is likely built in more recent years.

Joints at this location appear in good condition with only a few small sliver spalls at some transverse joints (as shown in Figure C.21). Smaller partial depth repairs are common, many of which are cracked and deteriorated. A faint map cracking pattern was apparent over a panel in which the tining was not very deep. This pavement was not cored.

C.4 TOWN OF FISHERS

Ten concrete pavement locations in Fishers were considered along 116th St, Allisonville Rd and 126th St that were built between 1997 and 2010 (as detailed in Table C.4). The condition of the joints in these pavement locations varied from good, sound joints to moderately severe deterioration at either the longitudinal or transverse joints, or both (as shown in Figure C.22). Reportedly the concrete was designed and pavements constructed using INDOT specifications that were current at the the time of construction (C. Tull, personal communication, 2011). Much of the materials and construction information was available so it provided an opportunity to compare performance with materials used. Reportedly Fishers uses "Clear Lane" deicers (C. Tull, personal communication, 2011).

After careful consideration and pavement site visits three pavements were cored as follows:

- Four cores were taken from 116th St that represented Phase 2 construction. This section was built in 2003 using vinsol resin as the air entraining admixture (AEA). The longitudinal joints in the EB lanes showed some deterioration but no deterioration was evident in the WB lanes.
- Six cores were taken from 116th St, Phase 3 which was built the following year in 2004 using the synthetic AEA, MicroAir. Both the longitudinal and transverse joints in the WB lanes showed some deterioration but no deterioration was evident in the EB lanes.
- Four cores were taken from Allisonville Rd, Phase 1, built in 2001–2002. This mix contained fly ash and the SB lanes was poured late in the season when it was cold and snowy. The NB lanes were poured the following year. Both the longitudinal and transverse joints in the SB lanes showed some deterioration but no deterioration was observed in the NB lanes.

It is assumed from the information provided that the concrete from 116th St Phase 2 contain similar materials as Phase 3 except for the AEA, that both concrete pavements were similar in design, constructed in the same manner and exposed to the same deicers and other environmental factors, except that Phase 2 has been exposed for 1 year longer than Phase 3. Some questions to consider during the examination and analysis of

TABLE C.4
Details of concrete pavements constructed in Fishers between 1997 and 2010.

# Cores	Phase	Location	Comments	Material and Contractor
	1[a]	116th St, EB	Very little steel used, no tie bars, no dowel baskets, cement treated subbase, skewed joints, faulted, check steel with locator (MIT Scan)	Builders, MicroAir AEA, gravel CA
		116th St, WB		
2 (L & M)	2[b]	116th St, EB	***EB shows deterioration*** along longitudinal joint only, transverse joint ok	2003: Berns & Berns: vinsol resin
2 (L & M)		116th St, WB	WB is not showing any deterioration	
3 (L, T, M)	3[a]	116th St, WB	***WB w/ joint deterioration*** in both longitudinal and transverse joints	2004: Berns & Builders: MicorAir AEA, Lehigh cement
3 (L, T, M)		116th St, EB	EB is in good condition	
	4[a]	116th St, EB	***EB w/ moderate to severe deterioration at joints,*** whereas WB is in good condition	2005: Berns & Builders: MicorAir AEA, Lehigh cement
2 "good" (L & M), 2 "bad" (L & M)		Allisonville Rd, Phase 1 (2001–2002)	**SB *Both longitudinal and transverse joints show deterioration*** in front of Jiffy Lube – late season pour, "snow flying." NB is not showing deterioration	Reith-Riley: Fly ash mix (fly ash from Rock Port)
	2	Allisonville Rd, Phase 2 (2005)	All joints are in very good condition. Minor jt deterioration in small areas that were hand poured	
	3	Allisonville Rd, Phase 3	Joints are in good condition. The intersection w/ 96th St in very bad shape	
		Allisonville Rd & 96th St	Hand poured, high w/cm, poor air void system, severe joint deterioration	
		126th St, EB	Good joints	
		126th St, EB	Good joints	

[a]Projects which used MicroAir AEA.
[b]Projects which used vinsol resin AEA.
EB = eastbound lanes, WB = westbound lanes.
116th St Phases 2-4 followed FHWA and INDOT Specifications.
126th St (1997): Joints in good condition but at time of site visit pavement being removed to reconstruct for improved geomatics.
126th St E of I-69 (2010): Joints in good condition after 2 winters, single saw cut.

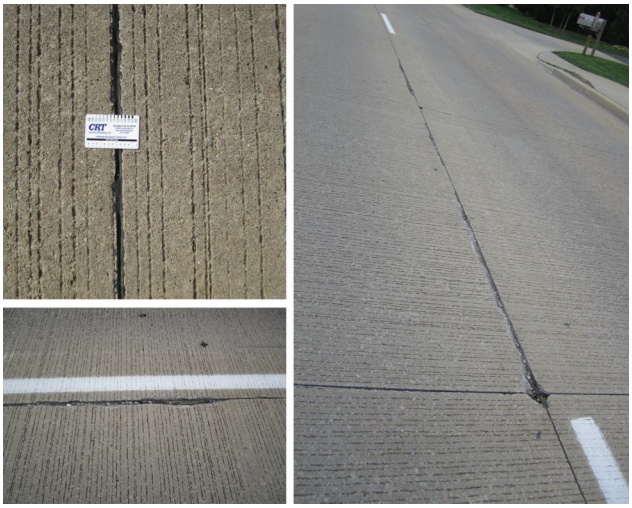

Figure C.22 Examples of joint conditions in concrete pavements examined in Fishers, IN.

these cores while comparing the two different phases of construction include the following:

- Confirm as best as possible that the materials used and mixture designs are similar
- Determine how the different AEA behaved under similar conditions in similar mixes
- Compare the original air void system created by vinsol resin to that created by MicroAir (e.g., spacing factor, specific surface, etc.)
- Compare the existing air void system in the vinsol resin concrete to that in the MicroAir concrete to determine if one is more prone to infilling.
- Determine, if possible, if the concrete material in the lanes that show deterioration is different from the concrete material that does not yet show deterioration at the joints.
- Determine, if possible, if the concrete construction or pavement structure in the lanes that show deterioration were different from that in the lanes that do not yet show deterioration at the joints.

Some questions to consider during the examination and analysis of cores from Allisonville Rd, Phase 1 include:

- What happens to the concrete matrix, microstructure, hydration products, etc., when a fly ash mixture is placed during cold weather and snowfall. How do these features differ from this same mixture placed under more ideal conditions? How do these features contribute to early joint deterioration?
- Are any of the unique features found in the mixture placed in late season paving similar to other concrete experiencing joint deterioration?

All the cores retrieved from Fishers allows for possible comparison of the influence of deicers. Reportedly Clear Lane is used by Fishers as a deicer and none of the INDOT cores examined under this project are believed to have been exposed to this deicer.

APPENDIX D: ELECTRICAL CONDUCTIVITY OF CONCRETE: THE EFFECTS OF TEMPERATURE, SATURATION AND AIR CONTENT

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Electrical conductivity of concrete : The effects of temperature, saturation and air content

Parth Panchmatia, Jan Olek and Nancy Whiting

This paper summarizes the results of investigations on the influence of changing temperature, saturation level and air content of hardened concrete on its bulk electrical conductivity. It was found that the temperature dependence of the bulk electrical conductivity of concrete closely follows the Arrhenius relationship. Formation factor, defined as the ratio of the pore solution conductivity to the bulk solution conductivity when the sample is saturated, was used to quantify the microstructure of concrete. Conductivity measurements were performed on concrete with four different air contents and it was found that the formation factor decreased with increase in the air content. This data was then used to derive a relationship between bulk conductivity of concrete and its air content. On drying, the tortuosity of the conduction path increases resulting in a decrease in the bulk electrical conductivity of concrete.

Keywords: *Electrical conductivity; formation factor; temperature; saturation, air content.*

INTRODUCTION

In general, concrete pavements offer good long term performance. However, in some cases, especially in the cold climate regions, premature joint deterioration has been reported as a growing durability problem. The symptoms of premature joint deterioration are not very obvious during the early inception stages. However, once the deterioration starts progressing it shortens the service life of the pavement and increases its maintenance costs. The problem appears to be strongly linked to the prolonged periods of moisture presence in the joints and the resulting increase in the saturation level of concrete. Once the concrete reaches critical saturation levels, which depends on the air void system present, it undergoes freeze-thaw deterioration which will further accelerate the process of ingress of water and thus facilitate accelerated freeze thaw damage [1,2]. For air entrained concrete, the saturation process takes significantly longer time compared to non-air entrained concrete [1,2].

To determine the cause of this premature joint deterioration, cores were obtained from both deteriorated as well as non-deteriorated sections of concrete pavements with the objective of conducting a series of materials and durability tests in the laboratory. The coring locations were selected based on the information collected from various divisions of the Indiana Department of Transportation (INDOT). Depending on the availability of data, the collected information included such items as the age of pavement, mixture composition, structural design parameters and type of deicer used.

Under service conditions, the temperature, pore solution concentration and the degree of saturation of concrete near the joints varies continuously. Measurement of electrical conductivity of concrete, which depends on

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concrete age (i.e. degree of hydration) [3,4], composition of the pore solution [5,6], moisture content [5,7], and temperature variations [8,9], may offer a convenient means to monitor these changes.

Based on the work done previously by INDOT in association with Joint Transportation Research Program (JTRP) on joint deterioration of concrete pavements in Indiana, USA [10,11], the research group was interested in obtaining construction, design, materials and deicing information for concrete pavements which demonstrated premature joint deterioration. Since most of this information was not easily available, the research team decided to prepare its own set of concrete mixtures which could be used to simulate the effects of individual variables on conductivity.

As mentioned above, the electrical conductivity of concrete depends on its age, composition of the pore solution, moisture content, and temperature. All of these parameters change when the concrete is subjected to freezing and thawing in the presence of deicing salt solutions thus making it difficult to interpret the test results. It is therefore essential to access how each of these parameters affects the conductivity of concrete individually before starting testing on samples where all the parameters change in tandem. This paper discusses the individual effects of temperature, saturation level and air content on the bulk electrical conductivity of concrete.

The Effects of temperature

In concrete, electrical conduction happens predominantly through pore solution [12]. Previous work has shown that Arrhenius relationship is applicable to concrete (see Equation 1) [8,9,13].

$$\sigma = Ae^{-\left[\frac{E_a}{RT}\right]} \quad \text{... (1)}$$

where, 'σ ' is conductivity of concrete (Siemens/meter), 'A' is the nominal conductivity at infinite temperature (S/m), 'E_a' is activation energy (kJ/mole), 'R' is the universal gas constant (8.314 kJ/mol/K) and 'T' is the absolute temperature (Kelvin). The above relationship has been successfully verified for temperature ranging from 0°C to 50°C [8,9,14]. The objective of this work was to determine whether the Arrhenius relationship is still applicable if concrete is exposed to temperatures below freezing and ranging from -18°C to 23°C (i.e. for cases where the low end of the temperature range is below freezing).

The Effects of Degree of Saturation

Air entrained concrete may still experience freeze-thaw deterioration if it is critically saturated (i.e. if the degree of saturation is greater than 85% [1]). Previous works have shown that there is a one-to-one relationship between the conductivity of concrete and its moisture content [7,15]. The tortuosity of the conduction path (defined as the inverse of pore connectivity) decreases with increased saturation level of the concrete, facilitating an increase in the measured conductivity. The present work examined the measured conductivity of concrete with degrees of saturation greater than 0.8, which is when the concrete is in danger of deteriorating due to freeze-thaw action [1,2].

The effects of air content

For a porous medium (like hardened cement pastes and concretes) the bulk conductivity (σ_1) can be modeled using Equation 2 [7,12,16].

$$\sigma_t = \sigma_0 \varphi \beta \quad \text{... (2)}$$

where 'σ_0' is pore solution conductivity (S/m), 'φ' is total liquid filled porosity, and 'β' is the factor representing moisture connectivity. The inverse of the product of pore connectivity and total liquid filled porosity (1/φβ) is called the formation factor [7,17]. Formation factor can be interpreted as a measure of the volume of the pores and their connectivity [7]. In other words, we could use formation factor to quantify the microstructure of concrete and therefore use the above concept to define a model connecting the air content of concrete to its measured conductivity. This work presents such a model which is applicable to concretes which have air contents ranging from 4% to 8%.

MATERIALS USED

This section provides information regarding materials used to produce different concrete mixtures which were utilized in this study.

Table 1. Chemical and physical characteristics of cement

Chemical composition	%
Silicon dioxide (SiO_2)	18.94
Aluminum oxide (Al_2O_3)	5.65
Ferric oxide (Fe_2O_3)	3.29
Calcium oxide (CaO)	63.20
Magnesium oxide (MgO)	3.13
Sulfur Trioxide (SO_3)	3.43
Loss on ignition	1.13
Sodium oxide (Na_2O)	0.34
Potassium oxide (K_2O)	0.78
Insoluble residue	0.35
Total alkali as Na_2O	0.86
Potential compound composition	
Tricalcium silicate (C_3S)	61
Dicalcium silicate (C_2S)	8
Tricalcium aluminate (C_3A)	9
Tricalcium aluminoferrite (C_4AF)	10
Physical characteristics	
Blaine fineness (cm^2/g)	3750
Autoclave expansion (%)	0.082
Air entrained (%)	9.8
Setting time (vicat) – initial/final (minutes)	112/225
Compressive strength (N/mm^2)	
1 day	19.24
3 day	28.00
7 day	32.34

Cement

Type I portland cement was used in all mixtures. Its physical and chemical characteristics are shown in Table 1.

Aggregates

INDOT approved No. 8 (d_{max} = 25.4 mm) aggregate from Delphi, IN and No. 23 natural siliceous sand was used in the production of concrete. The absorption and specific gravity of the fine aggregate and coarse aggregate were measured in accordance with AASHTO T 84 [18] and AASHTO T 85 [19] and are summarized in Table 2.

Table 2. Specific gravity and absorption of aggregates

	Absorption (% by weight)	Specific gravity
Coarse aggregate	0.011%	2.73
Fine aggregate	0.021%	2.56

Chemical admixtures

An air entraining agent was used. Use of water reducing admixtures was avoided because modern day super-plasticizers contain sulphonates or polycarboxylates which might alter the makeup of the pore solution and thus may affect the conductivity of concrete. In many of the older concretes being examined during this study water reducers were not used and the effects of temperature, air content and moisture are expected to be primary variables influencing the conductivity. The influence of additional variables, such as chemical admixtures, will be a logical next step for future work once the influence of the basic environmental variables are better understood.

MIXTURE PROPORTION AND SPECIMEN CONDITIONING

Mixture proportions

This section provides information on mixture proportions used to prepare test specimens for each of the three series of experiments (i.e. the evaluation of individual effects of temperature, saturation level and air content on the bulk conductivity of concrete). Initially, one particular mixture design (shown in Table 3) was adopted and used to prepare specimens to study the effects of temperature. This mixture design satisfied INDOT's requirements for concrete pavement. The concrete was mixed following AASHTO R 39 [20]. The air content of the fresh concrete was measured using a pressure meter following AASHTO T 152 [21]and was found to be 5.5% by volume. Temperature data loggers were embedded in cylindrical specimens (101.6 mm diameter and 203.2 mm height) during casting to monitor the temperature inside the concrete.

Table 3. Mixture proportions for testing the effects of temperature

Constituents	Amount (kg/m^3)
Cement (type I)	347.66
Water	149.50
Sand	812.79
Coarse aggregate	990.77
Air entraining agent	34.8 ml/m^3

The mixture with composition shown in Table 3 produced concrete with less than optimal workability (slump value of 25 mm). Since it was anticipated that the workability would be reduced further in the low air mixtures, this basic design was slightly altered by increasing the w/c ratio from 0.43 to 0.45. This modification increased the slump value to 50 mm. This altered mixture (composition shown in Table 4) was used for preparing samples for studying the effects of the degree of saturation on conductivity. The concrete was mixed following the procedure described in AASHTO R 39 [20] and cylindrical specimens (101.6 mm diameter and 203.2 mm height) were cast. The air content of the fresh concrete measured following the procedure described in AASHTO T 152 [21] and it was found to be 6% by volume.

Mixture with composition shown in Table 4 also was used as a base mixture to study the effects of air content (see Table 5). Table 5 shows the adjustments made to this mix design to obtain three more concretes mixture with the desired amount of air contents. These alterations were made by keeping the w/c and fine aggregate to total aggregate ratios constant. The concrete was mixed following AASHTO R 39 [20] and cylindrical specimens (101.6 mm diameter and 203.6 mm height) were cast. The air content of the fresh concrete was measured following the procedure described in AASHTO T 152 [21] and is summarized in Table 5.

Conditioning of test specimens

This section describes in detail the sample conditioning of concrete specimens used to study the individual effects of temperature, degree of saturation, and air content on the conductivity of concrete.

Table 4. Mixture proportions for testing the effects of saturation level

Constituents	Amount (kg/m^3)
Cement (type I)	347.66
Water	156.62
Sand	854.32
Coarse aggregate	1044.17
Air entraining agent	30.9 ml/m^3

Test specimens to study the effects of temperature

The specimens were demoulded 24 hours after casting and moist cured (23˚C and 100% RH) for 56 days. At the end of the curing period the specimens were removed from the moist room and were cut perpendicularly to their axis to remove about 1-inch thick slice from both the top and the bottom of each cylinder. The remaining part of each cylinder was then cut into three 2-inch thick disks. The curved surface of each disk was then sealed with epoxy. Once the epoxy cured the disks were then vacuum saturated following the procedure described in AASTHO T 277 [22]and (to ensure complete saturation) were kept in lime water until their mass stabilized. The saturated samples were then conditioned to 23˚C, 15˚C, 10˚C, 5˚C, 0˚C, -5˚C, -10˚C, -15˚C and -18˚C by placing them in the environmental chamber until the temperature inside a companion 50.8 mm thick concrete disk reached the desired temperature. The electrical resistance of each concrete slice was measured after each of the aforementioned temperatures was attained and stabilized (see description in section 4 of this paper). To prevent any loss of moisture due to temperature conditioning, the samples were sealed in an individual plastic bag each which was, in turn enclosed in another plastic bag containing some water in order to maintain the sample at 100% relative humidity.

Test specimens to study the effects of degree of saturation

The specimens were demoulded 24 hours after casting and moist cured (23˚C and 100% RH) for 28 days. Each

Table 5. Mixture proportions for testing the effects of air content on conductivity

	Mix 4%	Mix 5%	Mix 7%	Mix 9%
Constituents	Amount (kg/m^3)	Amount (kg/m^3)	Amount (kg/m^3)	Amount (kg/m^3)
Cement (type I)	347.66	347.66	347.66	347.66
Water	156.62	156.62	156.62	156.62
Sand	854.32	830.59	806.86	783.12
Coarse aggregate	1044.17	1008.57	984.84	955.18
Micro Air® AEA	27.1 ml/m^3	30.9 ml/m^3	42.5 ml/m^3	56.1 ml/m^3
Air content (AASHTO T 152)	4%	5.3%	6.7%	8.5%

cylinder was then cut into three 50.8 mm thick disk (after discarding the top and the bottom 25.4 mm thick slices) which were oven dried at 105˚C until their mass stabilized. The dried sample were then vacuum saturated following the procedure described in AASHTO T 277 [22] and were kept in lime water until their mass stabilized to ensure complete saturation. This condition was assumed to be a 100% saturated condition. The mass of the sample was recorded at 100% saturation level and the samples were dried to the desired mass values (which were theoretically obtained using completely saturated mass and dry mass of companion samples) to achieve saturation levels of 95%, 90%, 85%, and 80%. The electrical resistance was measured at each of those saturation levels.

The specimens to study the effects of air content

The specimens were demolded 24 hours after casting and moist cured (23˚C and 100% RH) for 28 days. Three 50.8 mm thick disks were cut out of the cylinder (after discarding the top and bottom 25.4 mm thick slices). The disks were then vacuum saturated following the procedure described in AASHTO T 277 [22] and placed in lime water until their mass stabilized to ensure complete saturation. The electrical resistance of the samples was then measured following the procedure described in section 4 of the paper. Air void analysis was performed on hardened concrete following the procedure described in ASTM C457 [23] to confirm that the air content was similar to what was measured in the fresh concrete.

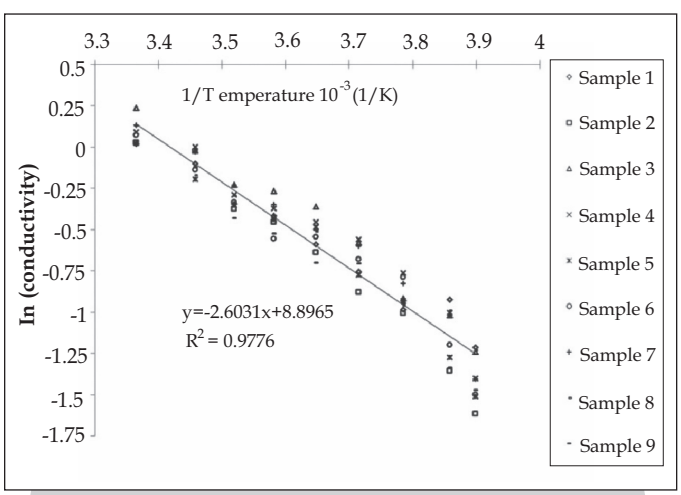

Figure 1. Plot of in(conductivity) vs.1/Temperature for nine specimens tested

MEASUREMENT OF ELECTRICAL RESISTANCE

The electrical resistance of concrete was measured using a Proceq® probe. The concrete sample was placed between two electrode plates and its resistance was measured. The resistivity of concrete was calculated by multiplying the resistance by the sample's geometry factor A/L (see Equation 3). The inverse of resistivity provided the value conductivity (see Equation 4).

$$\rho = R * \frac{A}{L} \quad \text{... (3)}$$

$$\sigma = \frac{1}{\rho} \quad \text{... (4)}$$

where 'ρ' is the resistivity (ohm-m), 'R' is the resistance (ohm), 'σ' is the conductivity (S/m), 'A' (m^2) and 'L' (m) are the cross sectional area and length of the sample respectively.

RESULTS AND DISCUSSIONS

This section presents the analysis and discussion of the results obtained from the experiments performed in the course of this study.

The Effects of temperature on the bulk conductivity of concrete

To assess the effect of temperature on the bulk conductivity of concrete, electrical resistance measurements were performed on a total of nine samples that were conditioned to nine different temperatures (as described in section 3.2.1). These results are plotted in Figure 1, and demonstrate that there exists a linear relationship between the natural logarithm of conductivity and inverse of the absolute temperature for temperatures ranging from -18˚C to 23˚C. On taking natural logarithms of both sides of Equation 1, we observe that the slope of the plot between natural logarithm of conductivity and the inverse of the absolute temperature is the activation energy for the process divided by the universal gas constant (R = 8.314 kJ/mol/K). Thus, the average activation energy, obtained by multiplying the slope of the straight line in Figure 1 by the universal gas constant, was found to be 21.64 kJ/mol. This value is close to the activation energy values of concrete obtained in other studies [8,24].

Table 6. The average conductivity values for concrete with different saturation level

Sample location	Average conductivity values (S/cm) of concrete and % decrease compared to fully saturated samples.				
	Saturation level 100 %	Saturation level 95 %	Saturation level 90 %	Saturation level 85 %	Saturation level 80 %
TOP	3.327	2.770	2.638	2.450	2.289
	% decrease	16.75	20.73	26.38	31.21
MIDDLE	3.187	2.653	2.509	2.360	2.194
	% decrease	16.75	21.27	25.94	31.17
BOTTOM	2.939	2.444	2.358	2.221	2.122
	% decrease	16.83	19.78	24.43	27.79

The effects of degree of saturation on the bulk conductivity of concrete

Previous works [7,15] have suggested that conductivity of concrete has a one-to-one relationship with its moisture content. Table 6 summarizes the changes in conductivity of concrete as it is dried from a state of complete saturation to a saturation level of 80%. Data presented in table are an average values obtained from four test samples. Slight differences in conductivity observed between the samples extracted from the top and the bottom of the 4"x8" cylinders could be attributed to segregation of the fresh concrete mixture. The samples from the bottom of the cylinder showed slightly lower conductivity (~10%) because they have higher percentage (by volume) of aggregate compared to the samples extracted from the top of the cylinder. Analysis of data from Table 6 indicates that conductivity decreases with drying for all samples. The percentage decrease in the conductivity is higher for the sample extracted from the top of the cylinder compared to the sample from the bottom of the cylinder for saturation levels less than or equal to 90%. This observation can be further explained by examination of Equation 2. Specifically, at the same saturation level, the total liquid filled porosity (φ) and the pore solution conductivity (σ_0) terms of the equation will be the same but the moisture connectivity (β) will decrease more rapidly for samples extracted from the top than those extracted from the bottom due to higher paste content in the top samples (which is a result of a partial segregation).

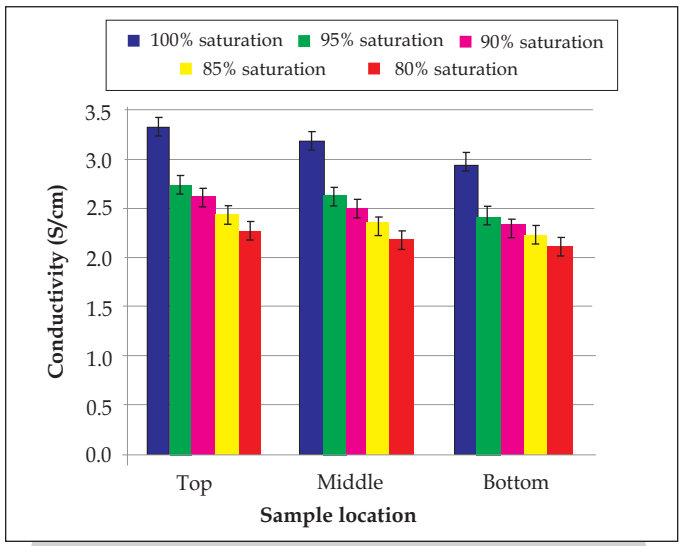

Figure 2. Changes in conductivity with changing saturation level of concrete.

Effect of air content on the bulk conductivity of concrete

Column 4 in Table 7 lists the average values of conductivity for concrete mixture with different amounts of entrained air content. It can be seen that, in general,

Table 7. Measured concrete conductivity values, measured pore solution conductivity value and the calculated formation factor

Mixture (1)	Air content (ASTM C231) (2)	Air content (ASTM C457) (3)	Average concrete conductivity, σ_t (S/cm) (4)	Pore solution conductivity, σ_0 (S/m) (5)	Formation factor, F (6)
A.C. 4%	4.00%	3.62%	1.26	4.7	0.037
A.C. 5%	5.30%	4.53%	1.39	4.7	0.034
A.C. 7%	6.70%	6.06%	1.66	4.7	0.028
A.C. 9%	8.50%	8.64%	1.30	4.7	0.036

as the air content increases the conductivity of the concrete increases except for the mixture with the highest (8.64%) air content. The increase in conductivity with an increase in the air content can be explained by the fact that in a saturated condition all air voids are filled with liquid which significantly decreases the tortuosity of the conduction path. Since change in the air content implies change in the microstructure of concrete, there is a need to quantify the extent of this change. As mentioned in section 1 of this paper, the quantification of the microstructure can be accomplished by calculating the value of formation factor (F) which is linked to both, the concrete conductivity (σ_t) and conductivity of the pore solution (σ_0) as shown in Equation 5.

$$\frac{\sigma_t}{\sigma_0} = \frac{1}{F} f(S) \qquad \text{... (5)}$$

where 'σ_t' is bulk conductivity of concrete; 'σ_0' is pore solution conductivity; 'F' is the formation factor; and 'f(S)' is a function of saturation which is equal to 1 in saturated condition.

Ideally the conductivity of pore solution should be determined using liquid squeezed out of concrete samples. However it was difficult to obtain a measureable quantity of pore solution from the mature concrete specimens. To overcome this problem, a series of paste specimen was prepared using the same cement and the w/c ratio as those used in preparation of concrete specimen. The paste specimens were mechanically squeezed to obtain pore solution needed to determine σ_0. The average pore solution conductivity measured at 28 days was 4.7 S/m. This value is considerably lower than the conductivity value of 14.63 S/m predicted for 100% hydrated concrete with w/c ratio of 0.45 by the model developed by the National Institute of Standards (NIST) [6]. The reason for this discrepancy may be related to the fact that specimens used in this experiment were vacuum saturated and soaked in lime water. Both of these treatments could have diluted the pore solution as a result of water ingress into the microstructure and leaching of alkali ions into the soak solution.

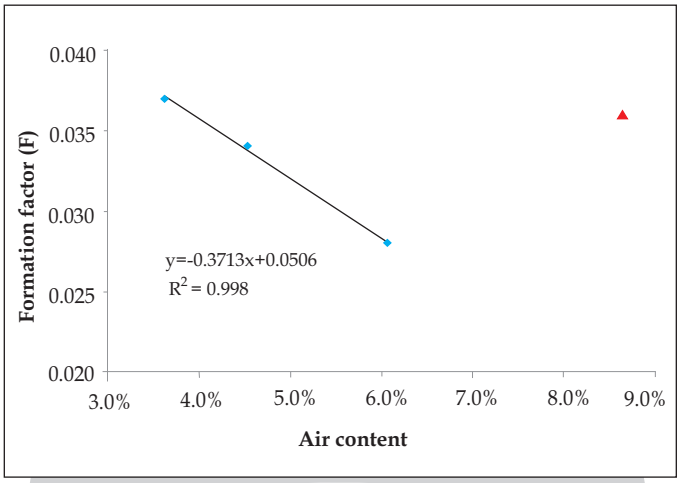

Figure 3. Formation factor vs. air content

The formation factors calculated using the obtained values of conductivity of the pore solution are also shown in Table 7. Figure 3 shows the plot relating the formation factor and hardened air content of concrete. The lower conductivity (higher formation factor) for mixture with air content of 8.6% may be due to the fact that at such high air content there might exists smaller air voids which are isolated and therefore are difficult to saturate completely. When unsaturated, these empty air voids will impede the flow of electrons essentially increasing the tortuosity of the conductivity path and reducing the bulk conductivity of those samples.

To verify the aforementioned assumption (presence of greater amounts of unsaturated air voids for mixture with high air content), the research team dried out samples at 105°C until their mass stabilized and tried to compare the actual mass loss to the theoretically expected mass loss. The theoretically expected mass loss should be equal to the sum of the evaporable water and the amount of water needed to completely saturate all the air voids. The mass of the cement (c) in the sample was estimated using the mixture proportions and the volume of the samples (which was same for all the samples). As all the mixtures had similar cement content and w/c ratios, the evaporable water, which was calculated by subtracting the non-evaporable water (0.24g/c) from the total water (0.45c), will be same for all the mixes. Analyzing the results given in the last column of Table 8, it is clear that the mix with air content of 8.6% had significantly lower amount of water in the air voids than those observed in mixes with lower air content. As a result, it should be expected that this particular mix will also have lower conductivity.

Table 8. The amount of air voids saturated at the time of measurement of resistance

Air content ASTM C457) (1)	Volume of the sample (cm3) (2)	Actual measured evaporated water (g) (3)	Theoretically calculated evaporable water (g) (4)	Water filling air voids (g) (5) = (3) – (4)	Theoretically calculated amount of water in air void at complete saturation (g) (6) = [(1)x(2)]/100	% of air voids actually saturated (7)=[{(6) - 5)} /(6)]*100
4.53%	720.74	79.35	52.6	26.75	32.65	81.92
6.06%	720.74	89.10	52.6	36.5	43.68	83.56
8.64%	720.74	92.40	52.6	39.8	62.27	63.91

For concretes with air content ranging from 4% to 8%, Equation 6 can be developed by combining the relationship between formation factor and air content (shown in Figure 3) and relationship between formation factor and conductivity (shown in Equation 5).

$$\frac{\sigma_t}{\sigma_0} = \frac{1}{0.05-(0.37*A.C)} \quad \text{... (6)}$$

where 'A.C.' is the air content of the concrete. This equation can be used to model the conductivity of saturated concrete with different air content (provided all other parameters are the same).

SUMMARY

The results of the present study clearly indicated that as the temperature of the specimens increased, their conductivity also increased. Arrhenius relationship (Equation 1) can be successfully used to model expected changes in concrete's conductivity as a function of its temperature for temperature range of -18°C to 23°C. The activation energy required to begin conduction process of a completely saturated concrete sample with w/c ratio of 0.43 was found to be 21.64 kJ/mol.

The study also revealed that as the degree of saturation of concrete increases, its conductivity increases. This increase was monitored for saturation levels ranging from 80% to 100% which is a typical range of moisture content associated with freeze-thaw damage. The influence of the degree of saturation on the conductivity of concrete can be predicted using previously published models [7].

For air contents ranging from 4% to 8%, the relationship between conductivity and air content of completely saturated concrete samples can be defined using Equation 6.

The results of the experiments reported in this paper also suggest that modeling of conductivity of concrete exposed to freeze-thaw conditions will require taking into account changes in conductivity resulting from temperature, degree of saturation and the air content.

The research team is currently working on developing a model which will relate the conductivity of concrete prepared in the laboratory to its temperature, air content, saturation condition, number of freeze-thaw cycles (in presence of different deicing solutions) for plain cement concrete and fly ash concrete. Based on the model (if successfully developed), the conductivity measurements performed on the concrete obtained from cores could provide an educated guess on the type of deicer used or age or any other information about the pavement which were not otherwise available. This could provide the research team with some of the missing information that might help identify causes of joint deterioration in concrete pavements.

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