Joint Deterioration in Concrete Pavements

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ABSTRACT

Concrete pavements located in cold climates have been experiencing premature joint deterioration. Entrapment of moisture in the joints saturates the surrounding concrete, rendering it susceptible to freeze–thaw damage. To identify and to isolate the variables that might be causing this localized deterioration, concrete cores were obtained from deteriorated and non-deteriorated sections of US 35, SR 38, and SR 3, located near Indianapolis, IN, and I-94, located near Michigan City, IN, USA. The visual evaluation of the condition of the pavement revealed that the drainage of the joints contributes significantly to their performance. Specifically, all deteriorated joint core holes drained poorly when compared with well-performing joint core holes or mid-panel joint core holes. Hardened air void parameters were determined following the procedure described in ASTM C457 and results for cores from deteriorated and non-deteriorated regions of the pavements were compared. The chemical and microstructural changes occurring in concrete were investigated using scanning electron microscope. Concrete panels with poor values of spacing factor and specific surface area were more prone to premature joint deterioration. Visual observation of coring sites on I-94 showed that unsealed joints performed better than sealed joints.

Keywords: concrete pavements, ettringite, Friedel's salt, infilling of voids, joint deterioration.

1. INTRODUCTION

In general, the service life of concrete pavements is assumed to be in the range of 30–40 years. However, some concrete pavements located in cold climatic regions have been experiencing premature deterioration at or near joints. The symptoms of this premature joint deterioration are often not initially visible at the surface [(as shown in Figure 1(a)], which makes early detection of their distress difficult. Typically, the distress is first seen at the surface of the pavement as microcracking near and parallel to the joint. With time, the cracking progresses, leading to significant loss of material around the joint [see Figure 2(b)]. Once this deterioration progresses, it decreases ride quality, increases maintenance costs, and disrupts traffic during maintenance.

Figure 1. (a) Deteriorated joint without any visible damage on the surface and (b) deteriorated joint with hints of underlying distress on the surface.

Joint deterioration is linked to prolonged presence of moisture in the joints resulting in saturation of the adjacent concrete (Arribas-Colon, del, Radlinski, & Olek, 2010). Once concrete reaches critical saturation levels (>85%), it becomes susceptible to freeze–thaw cracking (Li, Pour-Ghaz, Castro, & Weiss, 2012). Once cracked, concrete becomes more permeable, which leads to an increase in the rate of saturation of the surrounding region. This, in turn, makes the concrete around the joint even more susceptible to freeze–thaw deterioration.

Premature deterioration of joints in concrete pavements can be broadly classified as related to three issues: structural design, construction practices, and/or material properties. Joint spacing and saw cut depth are a few examples of structural design parameters that might contribute to joint deterioration. Poor construction practices, such as addition of excessive amount of water during placement, under or over vibration, or improper cleaning of sawed

Figure 2. (a) Initiation of deterioration with cracks parallel to the joint and (b) severely deteriorated joints.

joints before sealing, could also contribute to joint deterioration problem. The material properties of hardened concrete, such as the quality of air void system and diffusivity also significantly affect its ability to resist freeze–thaw damage (Kang, Hansen, & Borgnakke, 2012; Yang, Ge, Zhang, & Yuan, 2011).

Another factor that contributes to joint deterioration is the ability of the joint to drain any accumulated liquid. That ability depends on the width of the crack below the saw cut as well as the quality of the underlying drainage layer. The drainage layer has to be designed and constructed in such a way that it continues to drain water quickly over the design life of the pavement. Figure 3 shows a core hole that does not drain quickly. Sometimes, the pavement does not crack completely below the saw cut, and thus, any liquid accumulated in the joint will tend to saturate the surrounding concrete, thus accelerating its freeze–thaw deterioration.

Figure 3. Core hole with poor drainage.

Chloride-based deicers are extensively being used in cold climate regions to prevent excessive buildup of ice and snow on the pavement, and thus increase safety of the travelling public. Durability of concrete exposed to these deicers was extensively studied in the past and is well documented (Janusz, 2010). Recent studies (Taylor, Sutter, & Weiss, 2012) suggest that such changes in deicing practices like implementation of anti-icing and pre-wetting strategies, and increased use of aggressive (calcium- and magnesium-based

salts) deicers have contributed to the problem of joint deterioration. The damage to concrete exposed to chloride-based deicers manifests both physically (i.e., spalling, cracking, scaling) and chemically (deterioration of cement matrix; Olek, 2013). Recent studies on the effects of deicers on concrete deterioration conclude that calcium chloride is the most aggressive deicer, followed by magnesium chloride and sodium chloride (Jain, Janusz, Olek, & Jozwiak-Niedzwiedzka, 2011; Jain, Olek, Janusz, & Jozwiak-Niedzwiedzka, 2012).

To explore the factors contributing to premature joint deterioration, cores were obtained from both deteriorated and non-deteriorated sections of concrete pavements with an objective of conducting material and durability tests in the laboratory. A total of 22 concrete cores were extracted from three different pavements (US 35 in Muncie IN, SR38 in Newcastle IN, and SR 3 in Newcastle IN). All these cores were examined in the laboratory, and the results are presented in this article. In addition, a summary of results for 18 cores extracted from I-94 (near Michigan City, IN, USA) is also presented (Whiting & Olek, 2010). Selection of coring locations was conducted with the help of inputs collected from various divisions of Indiana Department of Transportation (INDOT).

2. SELECTION OF CORING LOCATIONS

A detailed questionnaire was prepared and sent to various units of INDOT to collect information about the occurrence of joint deterioration in Indiana. The questionnaire asked for such information as the age of pavement, concrete mixture composition, type of deicers and deicing practices used, and structural pavement design. Since only a limited amount of information was available, the research team conducted site visits to finalize pavement selections and coring locations. The generic layout of the coring pattern is presented in Figure 4. Starting with the next section of the article, the results of the analysis of cores obtained from US35, SR38, SR3, and I-94 will be presented and discussed.

Figure 4. Standardized coring pattern for selected pavements. A—Damaged transverse joint; C—damaged longitudinal joint; D—midspan core of deteriorated panel; E—undamaged transverse joint; F—undamaged longitudinal joint; Z: midspan core of nondeteriorated panel.

3. US35

Cores were obtained from US 35, located in Muncie, IN, USA from the location west of the intersection of US 35 and SR 3. At this location, the roadway consists of two undivided lanes of concrete pavement that were 10 years old at the time of coring. Upon casual observation, this pavement did not show visible signs of joint deterioration. However, upon closer observation, some raveling was detected along both transverse and longitudinal joints [see Figure 5(a)]. All the joints at this location showed minor raveling, and therefore, the so-called well-performing joints were actually joints that showed less raveling. The joints were sealed with silicone sealer and a backer rod. While coring, it was observed that core holes at the deteriorated joints did not drain well compared with core holes of midpanel or less deteriorated joints. Megascopic investigation of cores showed that the sawn surface (perpendicular to the pavement surface) of deteriorated joints showed signs of raveling beginning approximately 1" from the top of the pavement surface (region below backer rod) but that the less deteriorated joints had a smooth joint surface along the entire depth of the saw cut [see Figure 5(b)].

Figure 5. (a) Minor raveling of transverse (left) and longitudinal (right) joints on US35, Muncie, IN, USA. (b) Sawn joint faces of well performing joint (left) and deteriorated (right) longitudinal joint at US 35.

The air void system parameters of the concrete were determined using the modified point count method described in ASTM C457-2012 (Standard test method for microscopical determination of parameters of the air void system in hardened concrete). A scanning electron microscope (SEM) equipped with energy dispersive x-ray (EDX) capabilities was used to characterize the microstructure of concrete obtained from the cores.

Table 1 summarizes the air void analysis results of cores obtained from US 35. The total air content and spacing factor for all the cores (except for core C obtained from deteriorated longitudinal joint) did not satisfy the limits specified by Portland Cement Association for freeze–thaw durable concrete, that is, their spacing factor was greater than 0.008 and the air content was lower than 6% (Kosmatka, Kerkhoff, & Panarese, 2002). Despite core C showing the most deterioration and being associated with poor drainage observed during coring, the existing air void system appeared to have been adequate and would probably offer protection against freezing–thawing if the critical saturation could have been avoided. However air void system observed in all other cores examined for this pavement suggests that this section of US 35 is prone to future freeze–thaw damage.

Table 1. Air void analysis results for cores from US35.

Core		Air content (Vol. %)	Void freq. (/in.)	Specific surface (in. ² /in. ³)	Spacing factor (in.)
Det. Jt.	A	2.5	4.0	657	0.0148
	C	3.9	7.2	727	0.0084
Good Jt.	E	N.A.	N.A.	N.A.	N.A.
	F	2.9	4.2	593	0.0136
Midpanel	D	2.5	3.8	604	0.0149
	Z	2.9	4.2	590	0.0141

SEM analysis identified the presence of fly ash in the US 35 samples. Friedel's salt was detected up to a depth of 2" below the pavement surface in samples obtained from midpanel cores and up to a depth of 5" in all joint cores. In the top inch of joint cores, a relatively large amount of unhydrated cement paste was detected (see Figure 6). It is possible that the presence of sealer and backer rod at this location prevented access of water to unhydrated cement particles at the joint face, leading to the reduced hydration near the top of the pavement on the joint face. Another observation made during SEM analysis revealed that the infilling of voids with ettringite (see Figure 7) increases with depth and was more prevalent for joint cores compared with midpanel cores.

Figure 6. Unhydrated cement particles (seen as bright region under the SEM in backscatter mode) observed in US35 joint cores near the pavement surface.

Figure 7. In filling of voids with ettringite in joint core (5" from surface).

4. SR 38

Cores were obtained from two different locations on SR 38 (also known as Broad Street), New Castle, IN, USA. The concrete pavement type at this location is an in-town curb and gutter that demonstrates severe joint deterioration in the west-bound (WB) direction but no deterioration in the east bound (EB) direction. Cores were taken near the intersection of Broad and

Reddingdale street (R) and east of the intersection of Broad and 25th street (25). The location east of the 25th street showed minor joint deterioration (compared with Reddingdale street), which was restricted to the intersection of longitudinal and transverse joints (see Figure 8). The pop-outs visible on the surface of the pavement indicate the use of aggregate susceptible to freeze–thaw damage. Table 2 summarizes the observed drainage condition at all core holes at this location.

Figure 8. (a, b) Condition of pavement at location R (a, b) and at location 25 (c).

Table 2. Cores and core hole drainage conditions at SR 38.

Core	Drainage
R-A	Very poor; no change in water level for 15 min
R-C	Poor; took 10 min for water to drain out
R-D	Good; water drained in <2 min
R-E	Average; Water drained in 5 min
R-F	Good; water drained immediately
R-Z	Average; water drained in 5 min
25-A	Information not available
25-C	Good; water drained immediately
25-D	Poor; water drained very slowly
25-E	Information not available

The results of air void analysis for the cores from SR 38 are summarized in Table 3. The lapped concrete surface was treated following the procedure described by Poole and Thomas (1975) that stains sulfate ions to help to quantify the degree of infilling of voids with ettringite. In addition to staining ettringite, this procedure colored monosulfate making it difficult to differentiate paste from voids (possible source of error for air void analysis results). On comparing the results for cores from location R, it is evident that the deteriorated core (R-C) has the lowest air content and highest spacing factor in addition to demonstrating high degree of infilling. Similarly, for cores obtained from location 25, deteriorated core (25-C) has the worst air void parameters. Although the air void parameters for R-cores are much better than those obtained from location 25, that pavement showed higher degree of deterioration. This could be due to beneficial effects of fly ash (in terms of reduced permeability) used at location 25.

Table 3. Air void analysis results for cores from SR 38.

Core		Air content (Vol. %)	Void freq. (/in.)	Specific surface (in. ² /in. ³)	Spacing factor (in.)	Infilling (% air)
Det. Jt.	R-A	N.A.	N.A.	N.A.	N.A.	N.A.
	R-C	3.5	6.0	851	0.0100	18.87
Good Jt.	R-E	3.7	7.3	816	0.0080	3.57
	R-F	3.9	6.7	719	0.0090	5.00
Midpanel	R-D	3.7	6.4	717	0.0090	3.57
	R-Z	3.5	6.6	770	0.0090	3.70
Det. Jt.	25-A	N.A.	N.A.	N.A.	N.A.	N.A.
	25-C	4.0	2.4	235	0.0261	N.A.
Good Jt.	25-D	4.4	3.8	345	0.0156	N.A.
Midpanel	25-E	4.3	3.5	329	0.0167	N.A.

SEM analysis for R-cores revealed fly ash in the concrete with well-performing joints (EB) but no fly ash was detected in cores from the more deteriorated panels (WB). Friedel's salt was detected in all the joint cores up to the depth of 5 in (as shown in Figure 9). The amount of Friedel's salt found near the joint face was relatively constant up to the depth of about 3 in (which was the saw cut depth) but then decreased gradually with increasing depth. For midpanel cores, the occurrence of Friedel's salt deposits decreases with depth and is observed only up to the depth of 2 in from the pavement surface. Ettringite deposits compromised the air void system in both midpanel and joint cores. The degree of compromise is more prominent for deteriorated joint cores compared with nondeteriorated joints and midpanel cores. For example, higher degree of infilling is observed in core R-C beginning at depth of 1 in, whereas for core R-E, similar degree of infilling is noticed only after a depth of 2 in and for cores R-D and R-Z, it is observed at depths of 1.33² and 3², respectively. Calcium carbonate deposits were also detected in a few air voids at all depths for joint core samples and only up to 2" for midpanel core samples.

Figure 9. Friedel's salt deposit.

SEM analysis of core samples from location 25 showed presence of fly ash in all cores, which may explain the improved durability at the joints, despite the poor air void parameters. However, the low spacing factors and specific surface areas may eventually render this pavement susceptible to freeze–thaw deterioration, signs of which have already begun to show.

5. SR 3

SR 3, located on the west end of New Castle, IN, is a 4-lane undivided concrete pavement with turn lanes. The pavement south of the intersection of SR 38 and SR 3 shows severe deterioration of the longitudinal joint (see Figure 10), whereas no deterioration is apparent on the north side of the intersection. Cores were obtained from both regions of the pavement, and the observed drainage conditions at the core holes are summarized in Table 4. The core holes at the deteriorated joints (A and C) showed poor drainage compared with that for well-performing joints (E and F) and midpanel core holes.

Figure 10. Deteriorated longitudinal joint on SR 3, New Castle, IN.**Table 4.** Cores and core holes drainage at SR 3.

Core	Drainage	Other comments
113 A	Very poor	Broken in many pieces during coring
113 C	Very poor	Broken in many pieces during coring
113 D	Good	
115 E	Good	
115 F	Poor	
115 Z	Good	

The results of air void analysis for cores obtained from SR 3 are summarized in Table 5. The midpanel cores demonstrated better air void parameters compared with the joint cores, which can be explained by the infilling of the voids in the matrix near the joints with secondary products like Friedel's salt, ettringite, and calcium carbonate. The low spacing factors and specific surface area values for joint cores suggest that these so far well-performing joints may show signs of distress in the future.

Table 5. Air void analysis results for cores from SR 3.

Core	Air content (Vol. %)	Void freq. (/in.)	Specific surface (in. ² /in. ³)	Spacing factor (in.)
113 A	N.A.	N.A.	N.A.	N.A.
113 C	N.A.	N.A.	N.A.	N.A.
113 D	4.2	5.40	510	0.0098
115 E	4.4	4.75	413	0.0122
115 F	4.1	4.11	397	0.0132
115 Z	4.7	6.41	542	0.0088

6. FREEZE–THAW RESULTS FOR CORES FROM US 35, SR 38, AND SR 3

Freeze–thaw (FT) testing was performed on 4 in diameter by 1 in thick discs using method that is described in ASTM C666-2003 procedure A. The dynamic modulus of elasticity was measured using ultrasonic pulse velocity as described in ASTM C597-2009. For joint cores, two samples were obtained whenever possible, one at the joint face (FT1) and one about an inch away from the joint face (FT2; as shown in Figure 11).

Figure 11. Location of F-T samples for joint cores.

Table 6 summarizes the freeze–thaw results for samples extracted from cores obtained from US 35, SR 38, and SR 3. The freeze–thaw behavior for cores extracted from US 35 was similar for almost all samples except for core D (midpanel core), which cracked (into pieces) after exposure to 189 cycles. All the remaining FT1 samples showed a 40% reduction in modulus of elasticity after exposure to 328 freeze–thaw cycles. This reduction in the modulus of elasticity corresponds well with the air void parameters presented in Table 1. Another noticeable observation made was that the cracks often ran through the aggregate, indicating that the aggregates used may not be frost-resistant.

Table 6. Freeze–thaw testing results for cores from US 35, SR 38, and SR 3.

Pavement	Core		% Red. in E @ 328 cycles	# Cycles at failure
US 35	Det. Jt.	A-FT1	42.6	*D.N.B.
		A-FT2	16.7	D.N.B.
		C-FT1	41.3	D.N.B.
	Good Jt.	E-FT1	40.8	D.N.B.
		E-FT2	30.6	D.N.B.
	Mid panel	D		189
SR 38 R	Good Jt.	Z	37.3	D.N.B.
		E-FT1		95
		E-FT2		240
		F-FT1		120
	Mid panel	F-FT2		95
		D	28.4	D.N.B.
SR 38 25	Det. Jt.	Z	42.0	D.N.B.
		A-FT1		95
		A-FT2		246
	Good Jt.	C-FT1		205
		E-FT1		167
		E-FT2		300
SR 3	Good Jt.	E-FT1		95
		E-FT2		61
		F-FT1		61
		F-FT2		61
	Mid panel	D	31.8	D.N.B.
		Z	30.8	D.N.B.

*D.N.B., did not break.

All samples from SR 38, except those from midpanel cores, failed before reaching the target number of exposure cycles (300), indicating that both sections of the pavement are highly susceptible to future freeze–thaw deterioration.

The freeze–thaw results for SR 3 samples were in agreement with the air void analysis results given in Table 5. The midpanel samples, which had better air void system, demonstrated a 30% reduction in modulus of elasticity, whereas all joint samples deteriorated before completion of 100 freeze–thaw cycles. The well-performing joint cores showed very poor resistance against freeze–thaw exposure, implying that damage may be observed in the field in the future.

In general, it was observed that midpanel cores performed better than joint cores. Also, the FT1 samples deteriorated faster than the FT2 samples except for core F from SR 38 location R. FT2 samples being farther away from the joint are expected to have lower infilling of air voids compared with FT1 samples explaining the aforementioned observation.

7. I-94

Most of the important information, such as age of pavements, construction information, mixture design,

sub-base design, construction and conditioning, and deicing practices was not available for US 35, SR 38, and SR 3 pavements. Also, this information may have varied from pavement to pavement, making it difficult to make comparisons between them. On the contrary, the aforementioned information was available for I-94 located near Michigan City, IN. In addition, all sections of the pavement were paved under one construction project by the same contractor, using similar mix designs and materials. Finally, all sections of the pavement were exposed to similar environment, traffic loads, and deicing practices. Several sections of I-94 were in good condition except one that showed severe deterioration along the longitudinal joint. Due to the abundance of available information, study of this pavement provided a unique opportunity to identify factors causing joint deterioration. Additional details regarding this project are provided elsewhere (Whiting & Olek, 2010), and the summary presented here is provided to facilitate the comparison with the results obtained from US 35, SR 38, and SR 3.

Cores were obtained from a four-mile section between exits 34 and 40 on I-94 near Michigan City. This pavement is a six-lane divided highway with concrete shoulder on both sides of driving lanes. The EB lanes were constructed between September and December 1997, whereas the WB lanes were cast between June and October 1998. Because of late season construction for the EB lanes, fly ash was not used in sections that were placed after October 1997, whereas WB lanes had fly ash in all sections. All the joints were sealed using a backer rod and silicone sealer except for sections that were placed late in the year 1997 (EB direction).

Ten years after the construction of the pavements, it was noticed that all the joints in the EB lanes appeared to be in good condition. However, in the WB lanes, some of the concrete was deteriorating primarily along the longitudinal joints (see Figure 12). The deterioration took the form of spalling and loss of concrete near the joint resulting in widening of the joint to a few inches at the pavement surface. This deterioration was limited to the sawn longitudinal joint between the two outside driving lanes. No deterioration was observed in the construction joint between the innermost lane and the middle lane.

Figure 12. Deteriorated longitudinal joint, WB lanes of I-94 (Whiting & Olek, 2010).

A multitude of testing including air void analysis (ASTM C 457), SEM analysis, chloride concentration profiling (ASTM C 1556), and freeze–thaw testing was conducted on the cores obtained from different deteriorated and well-performing sections of this pavement.

Drainage of the base material was observed during core removal, and it was evident that the drainage was reduced at joints compared with midpanel locations (for both deteriorated and nondeteriorated joints). Analysis of base material established that the locations with a granular base drained better than locations where the base was no longer granular. Megascopic observation of the sawn joint cores revealed a difference between the joint face of the sealed joints compared with the unsealed joints. The joint face of the sealed double-cut, nondeteriorated joints showed raveling at depths below 2" from the top, whereas the joint face of unsealed single-cut joint was smooth for the entire depth of the saw cut (see Figure 13). Occasionally, the point where the sawn face changed from smooth to ragged coincided with the level of backer rod.

Figure 13. Cores from sealed (left) and unsealed (right) joints (Whiting & Olek, 2010).

The freeze–thaw analysis of samples extracted from the cores showed that concrete at the sawn joints had reduced durability factor compared with concrete from the midpanel cores, more so for the cores extracted from WB direction of the pavement. The air void parameters of all the cores extracted from the WB direction did not satisfy the traditional requirements for adequate freeze–thaw durability. On the contrary, all but one core obtained from EB direction had sufficient air entrainment to be classified as freeze–thaw durable. The SEM analysis revealed that specimens extracted from the joint cores showed higher infilling of air voids with secondary products such as ettringite and Friedel's salt than cores taken from the midpanels. Also, a higher degree of infilling was observed for cores taken from the deteriorated section compared

with nondeteriorated sections of the WB lanes. The mechanisms of infilling of air voids with secondary minerals such as ettringite, Friedel's salt, calcium carbonate are described in Section 8.

8. INFILLING OF VOIDS WITH SECONDARY MINERALS DEPOSITS

All the cores obtained from the aforementioned pavements showed some degree of infilling of air voids with secondary mineral deposits (ettringite, Friedel's salt, and sometimes calcium carbonate, as shown in Figures 9, 14, and 15, respectively). This infilling was more apparent in joint cores than midpanel cores.

Figure 14. Ettringite deposit in the voids.

Calcium carbonate deposits are a result of carbonation of the hydration products of cement, namely, calcium hydroxide and CSH gel. The atmospheric carbon dioxide reacts with moisture to form carbonic acid, which further reacts with calcium hydroxide or CSH to form deposits of calcium carbonate (Neville, 1997). The higher occurrence of calcium carbonate deposits observed for joint cores, especially deteriorated joint cores, could be associated to carbonation.

In a well-cured concrete, monosulfate (calcium aluminate sulfate C_4ASH_{12}) is present in abundance. In the presence of chloride, monosulfate is converted into Friedel's salt ($C_3A \cdot CaCl_2 \cdot 10H$) and gypsum (CSH_2 ; refer to Equation 1; Stark & Bollmann, 1979). Gypsum releases sulfate ions into the pore solution where they react with the remaining monosulfate in the presence of moisture to form ettringite ($C_6A\bar{S}_3H_{32}$; Equation. (2)). Figure 16 depicts the aforementioned process of formation of ettringite and Friedel's salt.

Figure 15. Calcium carbonate deposit in the void.

Figure 16. Mechanism of ettringite and Friedel's salt formation (adapted from Stark & Bollmann, 1979).

The higher degree of infilling of voids with ettringite and Friedel's salt observed in the joint cores can be attributed to the higher availability of moisture and chloride at these locations.

9. DISCUSSION AND SUMMARY

Cores both from deteriorated and nondeteriorated sections of the same pavements examined in this study provided insights into the causes of premature deterioration observed along the sawn joints.

A total of 22 cores were tested from US 35, SR 38, and SR 3. The results of 18 cores from I-94 examined as a part of another project (Whiting & Olek, 2010) also were summarized for comparison. Field observations of the drainage of core holes suggested that compared with well-performing joints, the deteriorated joint often correlated

with reduced drainage. Also, in general, midpanel core holes drained better than joint core holes. Joint opening is another important parameter required for adequate drainage, but this was not evaluated in this study.

Megascopic examination of cores obtained from I-94 showed a marked difference in the surface of the sawn sealed joints and sawn unsealed joints (Whiting & Olek, 2010). The saw cut surface of the sealed joints (even well-performing joints) were ragged below the backer rod. In contrast, sawn joint faces of unsealed joints were smooth all the way through their depths (Whiting & Olek, 2010). Similarly, a marked difference was observed between the sawn surfaces of deteriorated and well-performing joint cores obtained from US 35. Improper sealing of the joints (with silicon sealer and backer rod) may result in accelerated deterioration due to trapping moisture (which manages to enter the joint at locations where the seal is not tight) below the backer rod and critically saturating the surrounding concrete.

The air void parameters play a significant role in resisting freeze–thaw damage. The total air content (% vol.) of all noninterstate cores was below INDOT's current specified levels (5.5–8%). More important parameters, such as spacing factor and specific surface, were also not met for most cores (both deteriorated and well-performing cores). Current quality control (QC) procedure for ensuring adequate air content in pavement concrete measures the air before the concrete passes through the paver. QC needs to be altered to ensure that the target air content is met after concrete is placed. The spacing factors of well-performing joints were better than those for deteriorated joint cores for all pavements except for core C from US 35. In general, the midpanel cores had better air void system parameters than joint cores (maybe due to lesser infilling by secondary products). Ongoing research at Oklahoma State University to develop a technique to measure spacing factor of concrete in its fresh state has shown promising results (Leh, 2013). Such innovative techniques, if successful, would aid in field production of frost-resistant concrete.

For noninterstate core samples, all specimens (with few exceptions) failed the freeze–thaw test, and many of them broke and/or spalled into pieces before completing 328 freeze–thaw cycles. In general, the samples extracted closest to the joints performed the worst. This points to the fact that all joints (well-performing or deteriorated) are susceptible to freeze–thaw damage in the future.

For I-94 samples, all specimen passed the freeze–thaw test with minimum durability factor reported as 86. It was observed that the durability factors for joint cores were lower than that for midpanel cores and the lowest reported durability factor corresponded with the core obtained

from the well-performing joint in the deteriorated panel of I-94. In general, the air void analysis results correlated well with the measured freeze–thaw resistance.

The SEM analysis was used to characterize the matrix of the concrete cores. Concrete containing fly ash (cores from SR 38) demonstrated better joint performance. Various laboratory studies (Jain et al., 2011, 2012; Olek, 2013) also confirm that concrete with fly ash is more frost-resistant compared with Portland cement concrete without fly ash.

The degree of infilling of air voids appears to be higher for joint cores. Joint cores (deteriorated or not) show deposits of Friedel's salt up to depth of 5 in from the pavement surface as opposed to 2 in observed for midpanel cores. Carbonate deposits and ettringite infilling were more prominent for deteriorated joints than for well-performing joints, and it was least evident in midpanel cores for the same pavement. Carbonation and ettringite formation depend primarily on the availability of moisture, which is more prominent for deteriorated joints (since most deteriorated joint core holes drained poorly). The SEM analysis suggests that the degree of infilling of voids with ettringite increases with increasing distance from the pavement surface for all core samples. This may be due to decreasing chloride concentration with depth.

10. CONCLUSION

In conclusion, the following issues (from the materials point of view) need to be addressed to prevent premature distress along joints in concrete pavements:

1. The concrete should have adequate air content (and adequate air void parameters) after it is placed and finished. Innovative and field-friendly techniques to determine spacing factors and/or specific surface area of fresh concrete need to be developed and used during construction for quality control and assurance.
2. Concrete should be prevented from attaining critical saturation levels by providing a well-designed granular base and timely placed saw cut, which opens up to provide water a quick passage to the drainage layer.
3. The use of fly ash and other supplementary cementitious materials should be encouraged in order to densify the concrete matrix.
4. Unsealed joints perform better for I-94 but contradicting evidence exists that favor sealing of joints (Burke & Bugler, 2002). More durable and effective sealing techniques or materials that ensure a tight seal when exposed to extreme climatic conditions, and deicing chemicals needs to be developed.

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