Evaluating the Impact of Anti-Icing Solutions on Concrete Durability

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16. Abstract

There are concerns that the direct application of anti-icing solutions to dry concrete pavement surface and bridge deck prior to a snow event may result in higher amount of chloride penetration than the traditional method of applying rock salt to a wet, saturated concrete surface. The objective of this study was to quantify the impact of applied anti-icing solutions on dry concrete surfaces. To achieve this goal, this study completed a series of laboratory testing, accelerated field study at MnROAD, and analysis of historical performance data of Wisconsin pavements and bridges. Lab testing revealed that anti-iced concrete samples had roughly half the amount of material lost from surface scaling as the deiced concrete samples. Additionally, similar result occurred for the chloride content. Therefore, the concern of "anti-icing may lead to more chloride ingress to concrete" is invalid. This is likely due to two reasons: anti-icing uses less salt than deicing, and anti-icing had similar amount of chloride run-off with deicing (hence less chloride retained in concrete). Silane surface treatment provided a significant reduction in chloride penetration of around 50% for most conditions and epoxy effectively blocked the ingress of chlorides into concrete. In terms of the effect from traffic loads, the field results showed that in all cases the chloride content in the wheelpath was greater (9% at surface and 41% at the lower depth) than the non-wheelpath samples. It was therefore recommended for WisDOT to continue increasing the usage of liquid brine and anti-icing.

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Executive Summary

Several states including Wisconsin, have in recent years increased the application of anti-icing materials on pavements and bridges before a snowstorm event to prevent snow and ice from bonding to pavement/bridge surface. The benefits of anti-icing application have been well documented and include reaching bare pavement faster, improving pavement surface friction, requiring less salt, and being environmentally friendly. However, there are concerns that the direct application of anti-icing solutions to dry concrete pavement surface and bridge deck prior to a snow event may result in much higher penetration of anti-icing solutions than the traditional method of applying rock salt to a wet, saturated concrete surface. The direct application of anti-icing solutions may have significant long-term impacts on concrete durability due to the rapid ingress of the anti-icing solution.

The objectives of this study were to quantify the impact of applied anti-icing solutions on dry concrete surfaces, and to recommend countermeasures that would reduce the adverse impacts on concrete pavement and bridge deck durability.

To achieve these objectives, this study completed a series of laboratory tests, placed 12 concrete panels at MnROAD from 2021 to 2023, and analyzed historical performance data of Wisconsin pavements and bridges.

The laboratory test results proved that the high-quality Wisconsin concrete mixture exhibited very good freeze-thaw performance. The anti-iced concrete samples had roughly half the amount of material loss from surface scaling as the deiced concrete samples. Additionally, similar result occurred for the chloride content. Therefore, the concern of "anti-icing may lead to more chloride ingress to concrete" is invalid. This is likely due to two reasons: anti-icing uses less salt than deicing, and anti-icing had similar amount of chloride run-off with deicing (hence less chloride retained in concrete).

Silane surface treatment provided a significant reduction in chloride penetration of around 50% for most conditions and epoxy effectively blocking chlorides with low to no chlorides penetrated into concrete. When concrete sealer is applied at every 2 or 4 years on bridge decks, the analysis indicated that applying concrete sealers can help extend service life of bridge deck by around 1~2 years at stage 1-3 and reduce the number of overlays at stage 4. However, concrete sealer has to be applied frequently to maintain its long-term effectiveness.

In terms of the effect from traffic loads, core samples from MnROAD showed that in all cases the chloride content in the wheelpath was greater (9% at surface and 41% at the lower depth) than the non-wheelpath samples, confirming with the hypothesis that tire pressure greatly increases the ingress of chloride to concrete.

Therefore, we recommend WisDOT to continue increasing the usage of liquid brine by assisting more counties and municipalities with mixing equipment, storage facilities, tank trucks, and staff training. The current policy about deicing and anti-icing application rate should be updated based on recent studies such as Clear Roads Project 19-01. The practice of applying protective surface treatment to bridge decks is cost-effective and should be continued. The concrete samples tested in this project are high quality, proving the effectiveness of optimized gradation and supplementary cementitious materials (SCM).

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Chapter 1 Introduction

1.1 Background

Roadway snow and ice control strategies used in winter maintenance operations can be classified into four general categories: deicing; anti-icing; mechanical removal together with friction enhancement; and mechanical removal alone (NCHRP, 2004).

- Deicing is a snow and ice control strategy of removing compacted snow or ice already bonded to the pavement surface by chemical or mechanical means or a combination of the two.
- Anti-icing is a snow and ice control strategy of preventing the formation or development of bonded snow and ice to a pavement surface by timely applications of a chemical freezing-point depressant. Anti-icing can be initiated before a winter weather event or very early in the event.

Many laboratory studies and field evaluations have proved the benefits of anti-icing. For example,

1. Anti-icing reduced the effort required to remove compacted snow from the pavement surface (Cuelho & Harwood, 2012) (Hossain, Fu, & Olesen, 2014)
2. Anti-icing improved pavement surface friction after plowing, hence increased safety for the traveling public through reductions in accidents (Boselly, 2001) (Cuelho & Harwood, 2012) (Hossain, Fu, & Olesen, 2014). On urban roads, anti-icing was effective for reducing the majority of collision types and severities with reductions ranging from 8.7% to 49.8% on midblocks and between 5.4% and 13.0% at intersections (Gouda & El-Basyouny, 2020). On average, friction was 8.1% and road condition rating was 15.3% higher on the study route (with salt brine) than the control route (with solid salt) (Claros, et al., 2021).
3. Anti-icing can minimize the environmental impact of snow and ice control for air, water, and the roadside environment (Boselly, 2001) (O'Keefe & Shi, 2006)
4. Anti-icing can reduce costs of providing a specified level of service. Based on the surveys and interviews, savings of 10-20% of an agency's snow and ice control budget can be realized once fully implemented. Snow and ice control costs per lane mile can be reduced up to 50% (Boselly, 2001) (Hossain, Fu, & Olesen, 2014). Claros, et al. (2021) concluded that three was 23% reduction in salt use when using salt brine in comparison to solid salt.

The application of anti-icing has increased considerably in Wisconsin from 500,000 gal in 2006-2007 winter season to near 2,000,000 gal in 2016-2017 winter season (Xiao, Owusu-Ababio, & Schmitt, 2018). Minnesota's anti-icing usage also tripled in the last decade, from 1.8 million gal of salt brine in 2009-2010 season to 4.6 million gal in 2018-2019 season (MnDOT, 2019). Pennsylvania used 11.6 million gal of salt brine in 2018-2019 season, 55% increase from three years ago (7.5 million in 2015-2016 season) (PennDOT, 2020).

On the other hand, however, it is well proved that deicing and anti-icing materials cause damage to concrete infrastructure - either through deterioration of the concrete paste or corrosion of the reinforcing steel (Lee, et al., 2000) (Sakr, et al., 2020). Corrosion of reinforcing steel has typically been the primary deterioration mechanism and has been linked to the use of chloride-based snow and ice control chemicals such as $NaCl$, $CaCl_2$, and $MgCl_2$ (Shi, et al., 2009). Research (Lee, et al., 2000) has shown that different chloride-based snow and ice control chemicals can cause varying degrees of damage to concrete. This is mainly a result of specific chemical reactions between the associated cations (i.e., Mg^{2+}, Na^{+}, Ca^{2+}) with various phases of the cement paste.

As winter maintenance shifts from deicing to anti-icing, there are concerns that the direct application of anti-icing solutions to dry concrete pavement surface and bridge deck prior to a snow event may result in much higher anti-icing agent penetration than the traditional method of applying rock salt to a wet, saturated concrete surface. The direct application of anti-icing solutions may have significant long-term impacts on concrete durability due to the rapid ingress of the anti-icing solution.

1.2 Objectives

The objectives of this study were twofold:

1. Quantify the impact of applied anti-icing solutions on dry concrete surfaces, and
2. Recommend countermeasures that would reduce the adverse impacts on concrete pavement and bridge deck durability.

1.3 Research Methodology and Organization of Report

To achieve the aforementioned objectives, this research was composed of three major tasks:

1. Laboratory testing: A series of tests were conducted in controlled environments to compare the damage from deicing and anti-icing on typical Wisconsin concrete. The main difference between the two practices was whether chemicals are applied on wet or dry concrete surface.
2. Accelerated field study at MnROAD: Concrete panels from a Wisconsin project were placed at MnROAD where anti-icing and live traffic were applied from 2021 to 2023. The hypothesis was that hydrostatic pressure from tire load would lead to more penetration of anti-icing chemical and hence cause more damage to concrete.
3. Pavement and bridge management system data analysis: Historical performance data of Wisconsin pavements/bridges and winter maintenance records were analyzed. The hypothesis was that more damage would have occurred due to the increased use of anti-icing in Wisconsin if the concern was valid (i.e., anti-icing leads to more damage to concrete).

The report is written in five chapters. Chapter 1 is this introduction. Chapter 2 explains the experimental design and results of laboratory testing. Chapter 3 describes the process and results of accelerated field study at MnROAD. Chapter 4 is data analysis of the bridge management system. Finally, Chapter 5 summarizes this project and provides recommendations for WisDOT. To make the main report concise, the analysis of pavement management system is included in Appendix A. The literature review of topics pertinent to this project is attached in Appendix B.

Chapter 2 Laboratory Study

2.1 Introduction

The objective of the laboratory study was to investigate the main difference between deicing and anti-icing (i.e., liquid chemicals are applied on dry concrete before storm events during anti-icing) in controlled environment. The standard test methods ASTM C666 (freeze and thaw) and ASTM C672 (deicing chemicals) cannot simulate the unique condition of anti-icing, therefore, ASTM C666 freeze-thaw tanks were modified to follow the RILEM TC117 procedure. This chapter details the experimental design, materials used, procedure implemented, and test results.

2.2 Concrete Mixture Design and Specimen Preparation

In aim to use typical Wisconsin concrete in the laboratory study, all concrete specimens (cylinders and slab panels) were cast on site (contractor's concrete plant) using the paving concrete of WIS 23 Corridor Project (Fond du Lac to Plymouth) on Nov. 2, 2020. Table 2.1 shows the concrete mix design. Type A-FA concrete with optimized gradation and reduced cement content was used on the project, so concrete was first poured out of a regular truck, then the plant made another special batch without fly ash and keeping all other ingredient the same.

The temperature of the concrete was 56°F, slump was 1.25'', and air content was 6.8%. Specimens were cured on site for 48 hours during which temperature did not drop below freezing according to the temperature record. All specimens were transported to UW-Platteville Construction Materials Lab on Nov. 4, 2020. Cylinder samples were placed in lime-saturated water tank for curing. Slab specimens were covered with wet burlap and cured for 28 days. Specimens were then transported to UMKC for lab testing. Samples were sawn into appropriate sizes for different laboratory testing.

Table 2.1 Concrete Mix Design (per cubic yard)

	Unit	Type A-FA	Type A
Cement	lb	364	520
Fly Ash	lb	156	0
Stone 1	lb	1215	1215
Stone 2	lb	674	674
Sand	lb	1477	1477
Water	gal	24.0	25.2
Air entrainer (Polychem SA)	oz	5.5	5.5
Water reducer (Polychem 400 NC)	oz	15.6	15.6
Superplasticizer (Dynamon SX)	oz	16.1	15.6
w/cm ratio		0.39	0.40

Figure 2.1 (a) Plant-mixed concrete was poured into 4'*8' forms (b) consolidated and finished on site (c) sawcut into 2'*2' panels after 48-hour on-site curing (d) panels were transported to the lab and burlap cured for 28 days.

2.3 Testing Methods

A series of experiments were conducted to proceed stepwise through all aspects of freeze-thaw (FT) durability including scaling and drying/wetting. The experiments included:

- ASTM C666A Resistance of Concrete to Rapid Freezing and Thawing – Establish baseline FT performance.
- ASTM C672 Scaling Resistance of Concrete – Establish baseline scaling performance.
- NCHRP18-17/RILEM TC117 Capillary Suction and Freeze-thaw Test (CDF) – Using updated ASTM C666A equipment to create a one-dimensional gradient to allow more representative chloride movement and scaling assessment. First, establish baseline performance for FT and drying/wetting by testing in water. Then, assess differences in performance for variants tested in ASTM C672 deicing solution and anti-icing conditions.
- Tire Pavement Simulator (TPS) – Evaluate differences in chloride ingress in either deicing or anti-icing conditions with the application of tire pressure.

Chloride profiling of the various tests helps explain how the various test methods and conditions influence durability with the intention of linking laboratory performance to field observations from the MnROAD specimens.

2.3.1 Resistance of Concrete to Rapid Freezing and Thawing in Water (ASTM C666)

The baseline performance for the concrete mixtures was determined according to ASTM C666 procedure A, rapid freezing and thawing submerged in water. This test method provides a quantitative measure of resistance to the detrimental effects of freeze-thaw cycles. In addition to testing the unsealed control specimens for both concrete mixtures, silane or epoxy were applied to all six sides of the specimens. It should be noted that ASTM C666A is intended to verify coarse aggregate durability and to a lesser extent, air system quality (Taylor, et al., 2021). Testing of sealers is not conventional for this test method and the results section has a further discussion.

Samples were submerged in deionized water and cycled for 30 cycles per week. At the end of the 30 cycles saturated surface dry (SSD) mass and transverse fundamental frequency (ASTM C215) were individually measured and recorded. Furthermore, the mass loss of each sample was determined by filtering the material which had scaled during the previous set of cycles using a paper filter, followed by drying and weighing. The mass of the resulting dry scaled material was also recorded. The specimens were then returned to the freeze-thaw chamber, and the entire process was repeated for a total of 300 cycles.

2.3.2 Capillary Suction and Freeze-Thaw Test (CDF)

Sets of samples were also tested using a modified RILEM TC 117 (Setzer, Fagerlund, & Janssen, 1996) test adapted for the ASTM C666A equipment previously developed under NCHRP 18-17 (Taylor, et al., 2021) to better simulate field freezing and thawing conditions where freezing fluid is introduced one-dimensionally with freezing rate much slower at 2 cycles per day and a broader temperature range of 70°F and 0°F than ASTM C666A for 140 cycles. The CDF test was performed using three conditions: CDF-W (water), CDF-DI (deicer), or CDF-AI (anti-icer). An example cross-section is shown in Figure 2.2 where the broom-finished surface is oriented down, directly in contact with the freezing solution of choice.

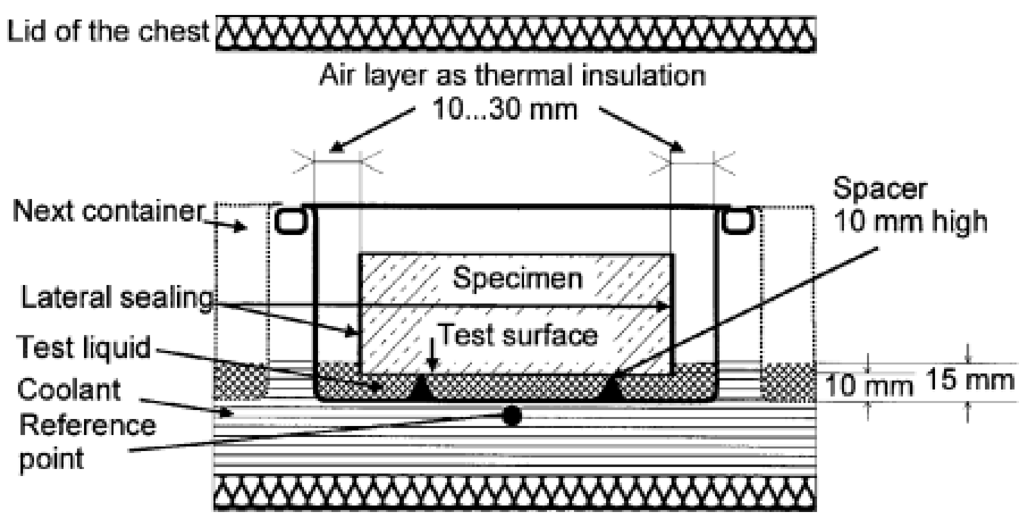


Figure 2.2 Cross-section of CDF Test setup (RILEM TC 117)

After sawing the samples were placed inside an environmental chamber at 23°C and a relative humidity of 50% for a period of 7 days to dry. Then a low-modulus and medium-viscosity epoxy coating (Sikadur-22 Lo-Mod) was applied to the lateral sides of the samples leaving the broomed surface and bottom uncoated. Once the epoxy was set, the desired sealer was applied to the broomed surface.

Experiment Procedure for CDF-W

The baseline CDF testing utilized deionized water as the freezing solution. Specimens were positioned with the test face side down and soaked for 7 days before obtaining the initial weight and frequency readings. Specimens were then cycled in contact with deionized water, refilling as needed. After completion of 14 cycles, the specimens were removed and placed in conditioning tank to allow equilibration to 70°F for a minimum of four hours before mass, scaled mass, and frequency were measured. Samples were then returned to the environmental chamber (23°C and 50% RH) for 3 days to dry. After drying, samples were conditioned in deionized water for 4 days prior to freezing and thawing cycles. In this fashion the test process involved one week of CDF testing followed by one week of drying and rewetting to capture both freezing and thawing and drying and rewetting.

Experiment Procedure for CDF-DI

The only notable distinction in the CDF-DI test procedure, when compared to the CDF-W procedure, is the utilization of a 3% sodium chloride NaCl solution for the freezing and thawing process. Apart from this, both procedures maintain an identical setup, wherein the solution is solely introduced from the bottom.

Experiment Procedure for CDF-AI

The CDF procedure was adapted to the application of liquid anti-icing brine to dry concrete. After the initial or weekly data collection, samples were placed in the environmental chamber to dry for 3 days. Then, the equivalent of 80 gallons per lane mile of a 23% NaCl solution was applied to the surface of the sample face to be tested using a dropper. The anti-icing solution was then allowed to dry for 1 day before conditioning prior to testing.

2.3.3 Sampling and Testing for Chloride Ions in Concrete

Chloride ion content was determined using acid titration according to ASTM C1152 and AASHTO T260. All the samples were ground to a powder with caution to prevent contact of the material with hands, or other source of body perspiration or contamination. As such, all sampling tools, such as spoons, clear storage container, and sieve, were washed with alcohol and dried before use. A diamond profile grinder was utilized to obtain the specimens. Per AASHTO T260, the surface layer to 1.6mm was removed, depth #1 reports average chloride content collected from depths ranging from 1.6mm to 13mm, and depth #2 from depths ranging from 13mm to 25mm. Figure 2.3 shows the setup used to extract chloride powder samples from a CDF beam. Sanitized spoons were utilized to collect the resulting powder and transfer it to the appropriately labeled container. Subsequently, a vacuum cleaner was used to ensure the complete removal of all powder from the corresponding layer before starting the subsequent layer drill process as shown in Figure 2.3.

Figure 2.3 (left) Profile grinder and CDF sample prior to collecting powder sample for chloride analysis, (right) Collecting grinded powder

2.3.4 Resistance of Concrete to Chloride Ion Penetration

The 90-day salt ponding test followed AASTHO T259 with analysis following AASTHO T260. Figure 2.4 shows two of the samples undergoing the 90-day ponding using the 3% NaCl solution.

Figure 2.4 90-day ponding arrangement

2.3.5 Lab simulation of the influence of tire pressure on chloride penetration

A test was developed to link the laboratory chloride diffusion under drying and wetting and freezing and thawing with the field specimens which also experienced traffic. The Midwestern Regional Climate Center (MWRCC) was used to determine the test parameters using average event data from Madison, WI. Madison averages 25 snow events per year with an average of 50.7 inches of snow per year and temperature of 25°F (MWRCC, 2023). Using the National Weather Service (NWS) reported snow ratio of 12:1 for the upper Midwest, this would result in a total of 4.23 inches of snow-based precipitation from October to April (NWS, 2023).

Specimens were dried in the environmental chamber prior to testing. Following this initial stage, the samples were subsequently transferred to a freezer maintained at 25°F. Samples received one of three treatments. The first group served as control samples, without any chemical treatment and only received snow. The second group, referred to as the DI samples, received an application of rock salt (NaCl @ 250 lbs/lm) after the snow simulation. While the third group, denoted as the AI samples, received an application of anti-icing solution (80 gallons/lm @ 23% NaCl) prior to snow application. To prevent unwanted snow melting, only one sample at a time was removed from the freezer. Using a snow cone maker and ice cubes, artificial snow was produced and 33 grams applied onto the surface of the designated face of each sample, as shown in Figure 2.5. The artificial snow density was measured and calculated as $\gamma(\text{snow})=0.98$ g/ml.

Subsequently, the samples were compacted with the layer of snow by employing a compression testing equipment configured with a car tire setup, exerting a pressure of 75 psi to simulate the vehicle action on concrete pavement. The pressure was selected using the maximum AASHTO HS-20 loading of 32 kips/axle and 120 in^2 rectangular contact area.

Figure 2.5 Sample before, during, and after tire compression

The DI samples received solid NaCl applied to the surface of the snow prior to tire compression and the AI samples received the liquid brine directly to dry concrete which was allowed to soak into the concrete in the freezer for 24 hours prior to application of snow and tire pressure as shown in Figure 2.6. The lowest effective temperature for a 23% NaCl solution is 15°F so the solution was liquid in the freezer. After compression of the snow with the tire setup, the samples were returned to the freezer for one day at a temperature of 25°F. Each sample was placed inside an individual plastic container. Afterwards, the samples were removed from the freezer and allowed to melt in the lab, ensuring they were covered with plastic to prevent evaporation, for 4 hours.

Figure 2.6 Application of solid or liquid NaCl

After the melting process was complete, the runoff water was collected from each plastic specimen and the volume was recorded. The liquid was collected for chloride analysis at 1, 5, and 25 cycles. The samples were left to dry for one day before being reintroduced into the freezer in preparation for the next cycle. Each cycle corresponded to a particular snow event. The culmination of 25 cycles simulated a year. Upon completion of 25 cycles, chloride analysis was performed at the two depths according to AASHTO T259.

2.4 Results Analysis

2.4.1 Concrete Properties

Compressive strength was tested on cylinder samples following ASTM C39 at UW-Platteville Materials Lab. Three replicas were tested for each concrete. As shown in Figure 2.7, the 28-day compressive strength *f'c* for type A-FA concrete is 6,484 psi while the type A concrete is only 3,792 psi. In other words, *f'c* for type A-FA is 70% greater than type A concrete. Both concrete types keep gaining strength as curing continues, and type A-FA keeps overperforming than type A.

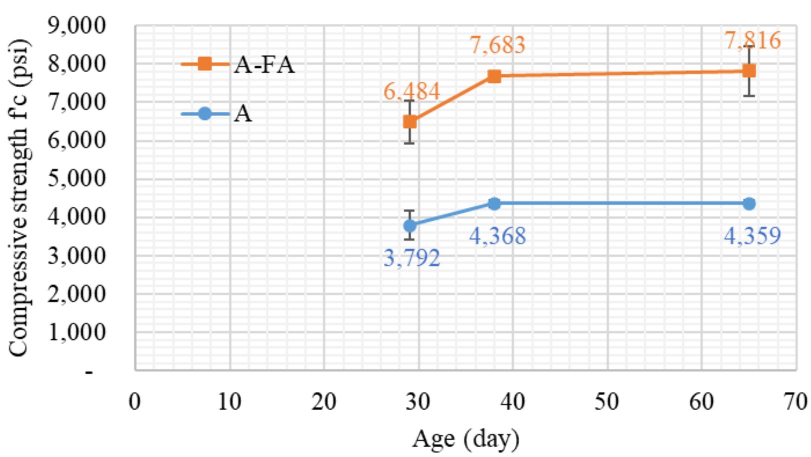


Figure 2.7 Compressive Strength

After concrete specimens being transported to UMKC, the air-void system was determined through microscopical analysis following ASTM C457. Results are listed in Table 2.2. The air content is slightly lower than the spec requirement, but note that the air content of fresh concrete (6.8%) did meet the spec. Similar difference has been reported by Khayat & Nasser (1991) (air content in fresh concrete can be 0.5 to 1.5% greater than that actually present in hardened concrete), and Pham & Cramer (2019) (the difference between air content measured by fresh concrete and hardened concrete were within ±2%). Spacing factor and specific surface area were both within the typical range and on the superior end, indicating good quality of the Wisconsin concrete.

Table 2.2 Test results on hardened concrete

	Type A Concrete	Type A-FA Concrete	Typical Range
Air Content	4.6%	5.2%	5.5~8.5%[a]
Spacing Factor (in.)	<0.006	<0.006	0.004~0.008[b]
Specific Surface Area (in^2/in^3)	823	853	600~1000[b]
Surface Resistivity (kΩ·cm)	25.2	32.6	

Note: [a] Per WisDOT Standard Spec 501.3.2.4.2 Air Entrainment. [b] Per ASTM C457/C457M-23

2.4.2 ASTM C672 Deicer Scaling Results

The ASTM C672 deicer salt scaling visual rating results are shown in Table 2.3. ASTM C672 is the most utilized test method for assessing concrete scaling. In general ASTM C672 is considered a coarse assessment of durability as the quality of finishing in the field often playing a significant role in scaling. All concrete samples had excellent deicer rating with only the untreated control specimens from the A-FA mixture showing any notable scaling. Figure 2.8 shows representative specimens before testing and Figure 2.9 the same specimens after 50 cycles. The A-FA control experienced a small amount of scaling with the silane and epoxy-treated versions providing improved scaling resistance.

Table 2.3 ASTM C672 Visual Rating of Concrete Scaling (0 good-5 bad)

Cycle	A-Control	A-Epoxy	A-Silane	AFA-Control	AFA-Epoxy	AFA-Silane
0	0.0	0.0	0.0	0.0	0.0	0.0
5	0.0	0.0	0.0	0.5	0.0	0.0
10	0.0	0.0	0.0	0.5	0.0	0.0
15	0.0	0.0	0.0	0.5	0.0	0.0
20	0.0	0.0	0.0	1.0	0.0	0.0
25	0.0	0.0	0.0	1.3	0.0	0.0
30	0.0	0.0	0.0	1.5	0.0	0.0
35	0.0	0.0	0.0	1.5	0.0	0.0
40	0.0	0.0	0.0	1.5	0.0	0.0
45	0.0	0.0	0.0	2.0	0.0	0.0
50	0.0	0.0	0.0	2.5	0.0	0.5

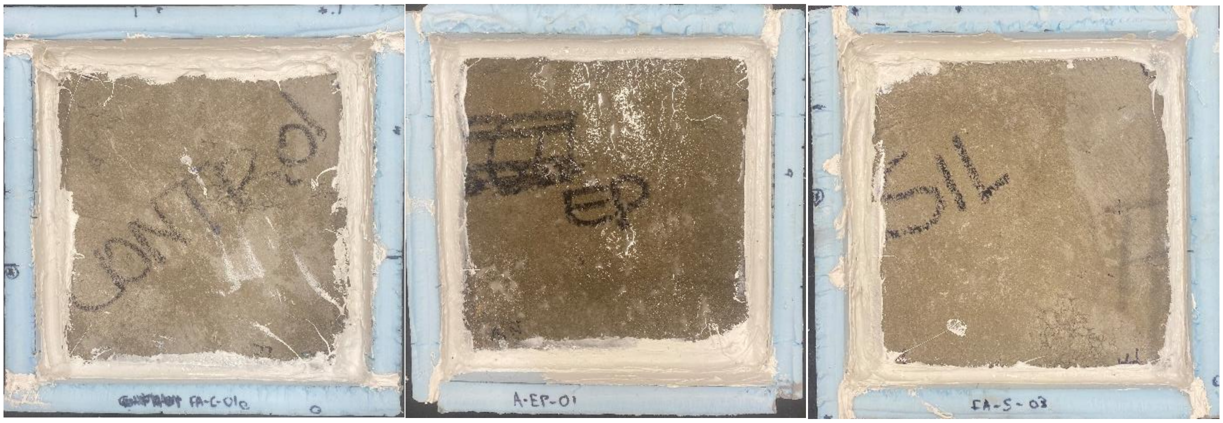


Figure 2.8 Representative Deicer Scaling Specimens Before Testing (from left to right: A-FA Control, A Epoxy, A-FA Silane)

Figure 2.9 Representative Deicer Scaling Specimens After 50 Cycles (from left to right: A-FA Control, A Epoxy, A-FA Silane)

2.4.3 ASTM C666A Freeze Thaw Results and Discussion

The relative mass and relative dynamic modulus results for the ASTM C666A test are shown in Figure 2.10 and Figure 2.11, respectively. ASTM C666A is conventionally used to assess the freeze-thaw (FT) resistance of plain concrete and most sensitive to coarse aggregate durability and concrete air system, although there is much debate surrounding the appropriateness of the submerged conditions, rapid freezing and thawing rate, and 300 cycles (Taylor, et al., 2021). Observing the untreated control specimens for the A and A-FA concrete with the A concrete having a durability factor of 88 and the A- FA of 98, indicating good performance for both with A-FA having better performance.

The subsequent discussion pertains to the inappropriateness of ASTM C666A for evaluating sealers supporting the use of other employed test methods for sealer evaluation. The relative mass of the specimens is shown in Figure 2.10 where the typical behavior is for the sample mass to increase as microcracking increases absorption, followed by a decrease in mass when solid materials scale from the surface. In this fashion the measured sample mass can increase even though material has been lost because the amount of extra water absorbed in the new cracks is greater than the mass of material lost. The samples coated in epoxy gained a small amount of mass as the epoxy is not completely impermeable.

The performance of silane is important to discuss in particular. Silane is a hydrophobic, pore-lining sealer, which allows movement of water vapor. In the sealer sales literature this aspect is often described as breathable. When silane is applied to all surfaces of the concrete and placed in submerged FT conditions, the degree of saturation increases as water vapor is allowed to move into the concrete but not move out. In ASTM C666A testing the samples are only brought to saturated surface dry (SSD) condition for testing and immediately placed back into the water, preventing any drying to occur. As observed in Figure 2.10, the sample mass increased rapidly until critical saturation was reached and deterioration rapidly followed. While ASTM C666A is often considered an overly harsh or conservative test for conventional concrete, the lack of drying or gradient for moisture movement suggest it is especially impractical for sealers.

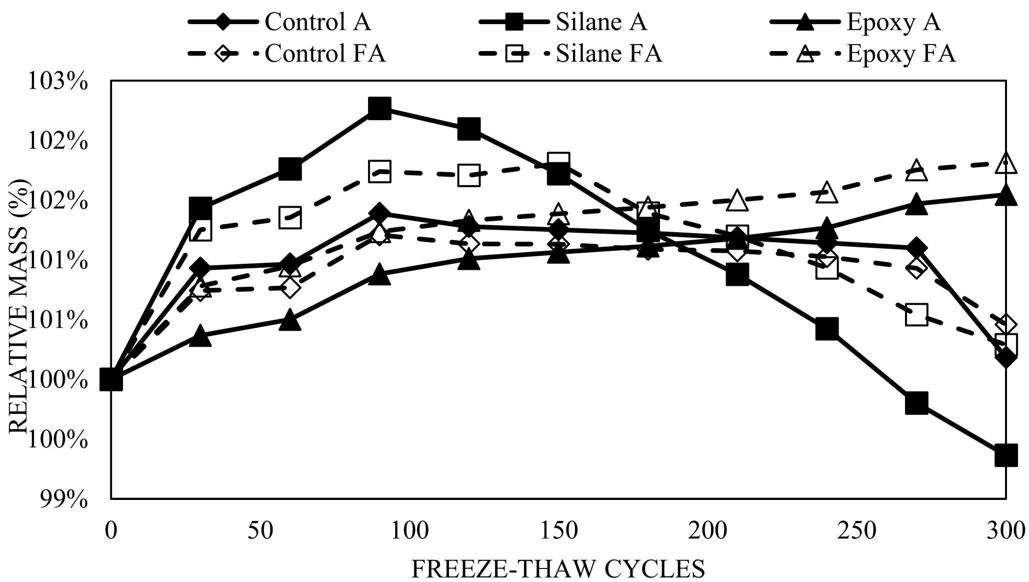


Figure 2.10 ASTM C666A Relative Mass of Concrete Beams

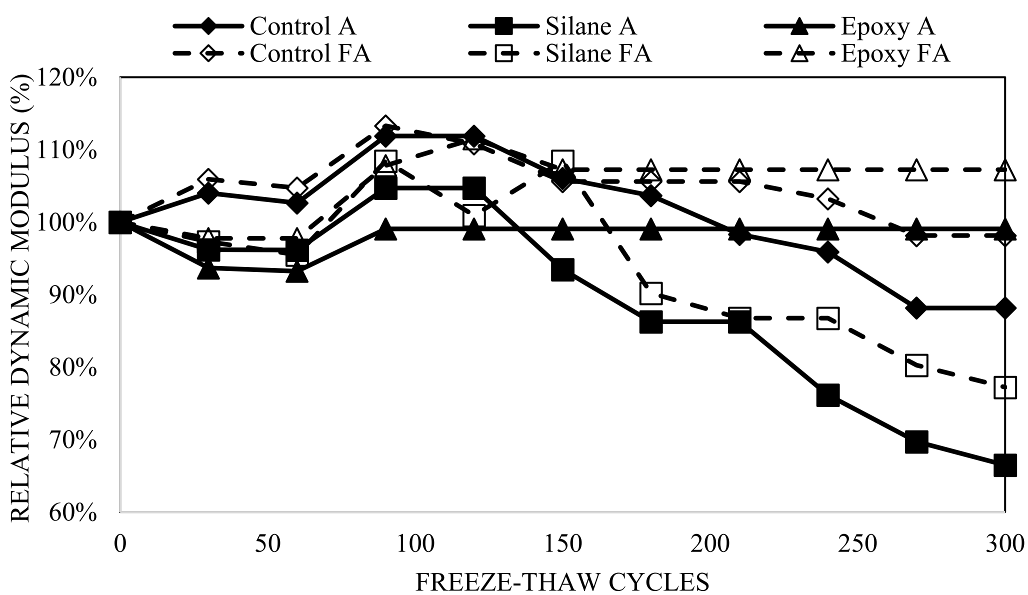


Figure 2.11 ASTM C666A Relative Dynamic Modulus

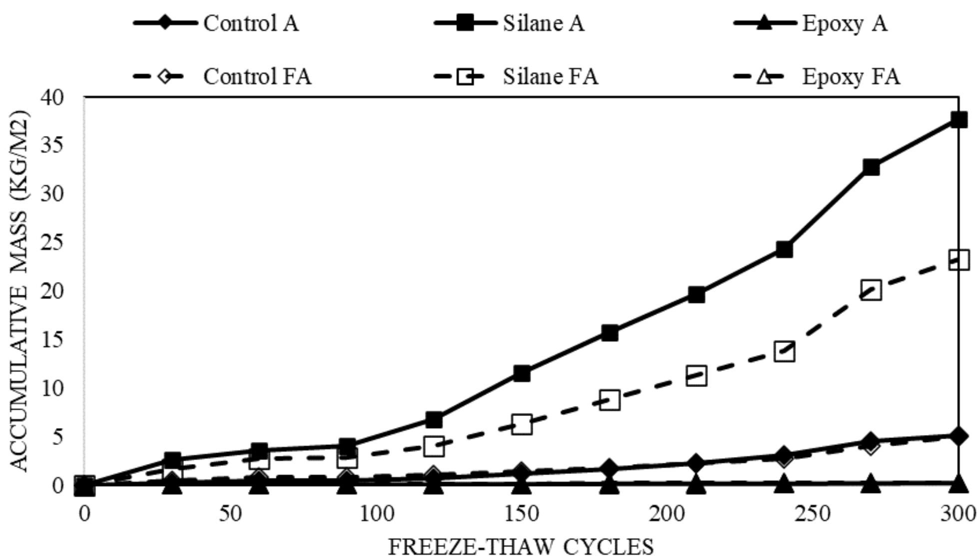


Figure 2.12 ASTM C666A Accumulated Mass Loss from Scaled Material

2.4.4 Modified RILEM TC117 CDF Freeze Thaw Results and Discussion

To address the observed shortcomings of ASTM C666A with respect to the ability to test sealers and deicing practices, NCHRP 18-17 was commissioned to determine a more appropriate and flexible test method for evaluating these aspects on concrete. RILEM TC117 is a test primarily used to evaluate deicer scaling, but has been noted for more realistic, one-dimensional introducing of freezing-fluids with a drying gradient and more realistic FT cycle time of two per day. In NCHRP 18-17 these aspects were adapted to the common ASTM C666A equipment and dynamic modulus testing for capillary exposure to deicing fluid (CDF) test. Figure 2.13 and Figure 2.14 show the dynamic modulus and scaled mass results for the CDF testing performed with deionized water as the freezing fluid (CDF-W). All samples had good performance with a small amount of material scaled from the surface. For context, the Ministry of Transportation in Ontario uses a cutoff value of 0.80 kg/m^2 of scaled material to separate acceptable from unacceptable performance with Quebec using a stricter 0.50 kg/m^2 (Taylor, Hooton, & Vassilev, 2012). The observed values less than 0.04 kg/m^2 indicate very little scaling.

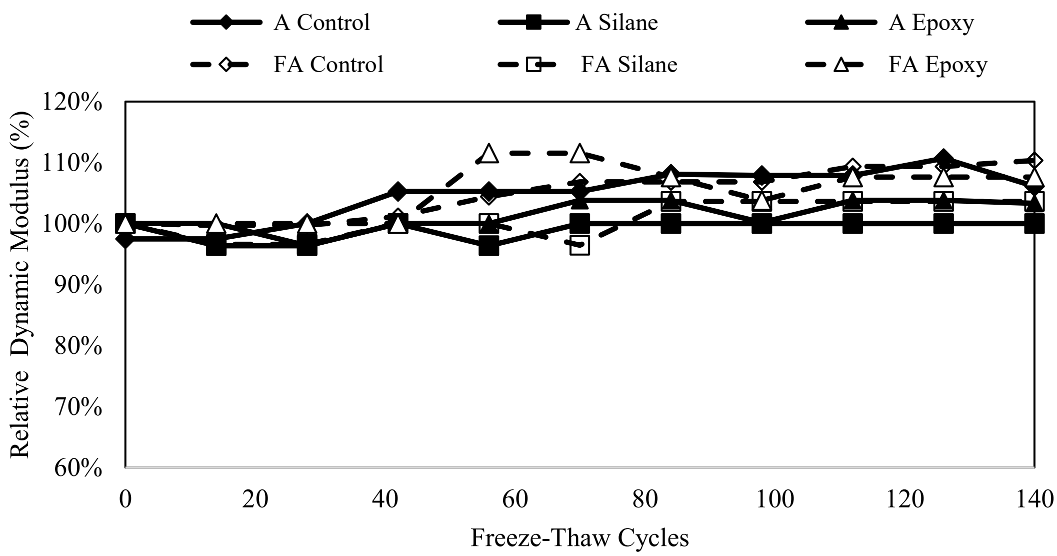


Figure 2.13 CDF-W Relative Dynamic Modulus

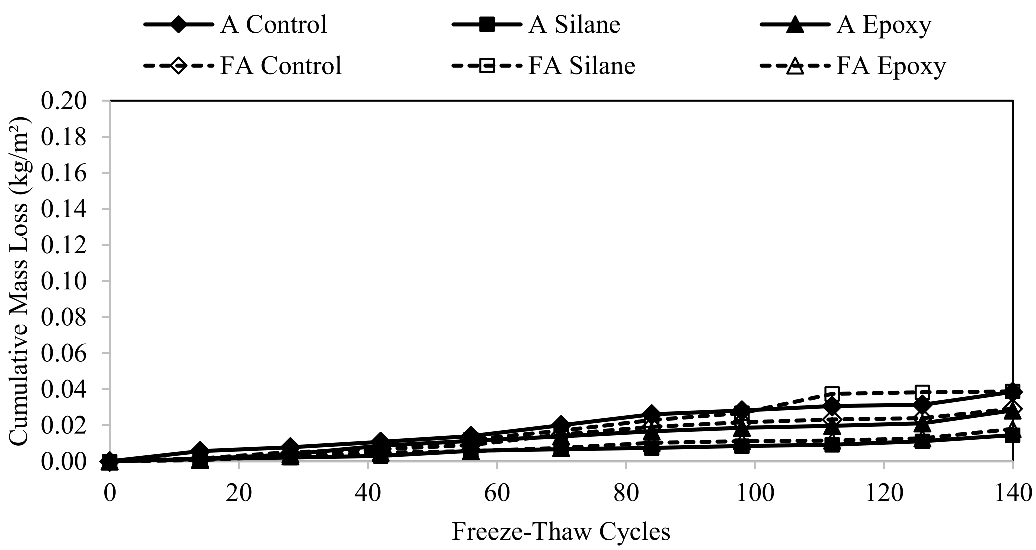


Figure 2.14 CDF-W Cumulative Mass Loss from Scaling

Figure 2.15 and Figure 2.16 present the dynamic modulus and scaling results for samples where 3% NaCl was used as the freezing fluid (CDF-DI) and Figure 2.17 and Figure 2.18 for the samples receiving the 23% NaCl liquid anti-icer brine (CDF-AI). In general, all samples had good performance. It should be noted that when CDF testing uses salt treatments, the dynamic modulus will increase slightly as the test progresses as salt penetrates the specimens and either reacts to form other solid products or crystallizes. While in the short-term this provides strengthening to the concrete, the continued introduction of salt will have many negative consequences to durability. Both the CDF-DI and CDF-AI treatments had significantly higher scaling than the CDF-W, water control. The scaling from the deicer samples (CDF-DI) was roughly double that of the liquid brine anti-icing (CDF-AI) samples. This is consistent with the literature showing around 3% NaCl resulting in the most significant durability damage (Valenza & Scherer, 2006). While the samples tested in the deicing fluid had higher scaling than those tested in water, material scaled was well below the 0.80 kg/m^2 cutoff and consistent with the ASTM C672 data showing high resistance to scaling.

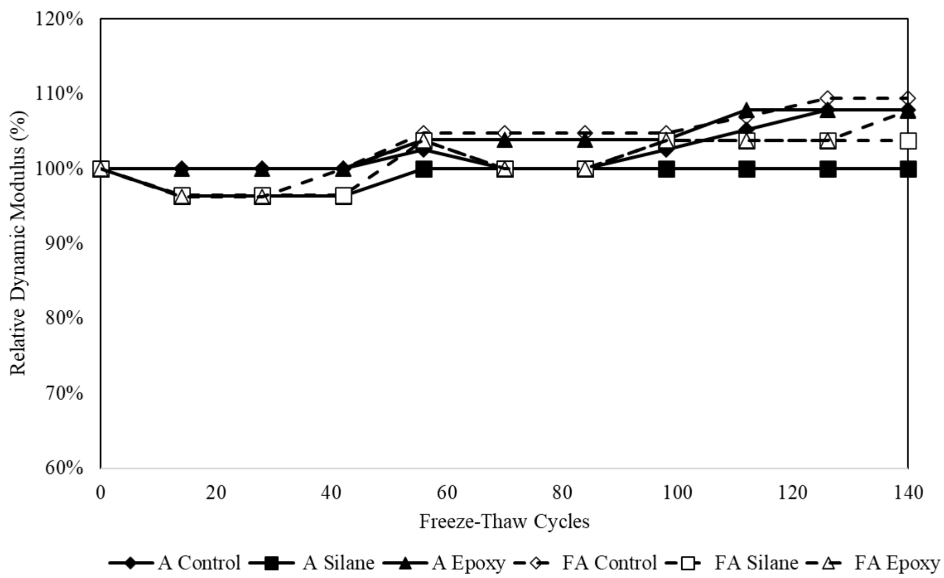


Figure 2.15 CDF-DI Relative Dynamic Modulus

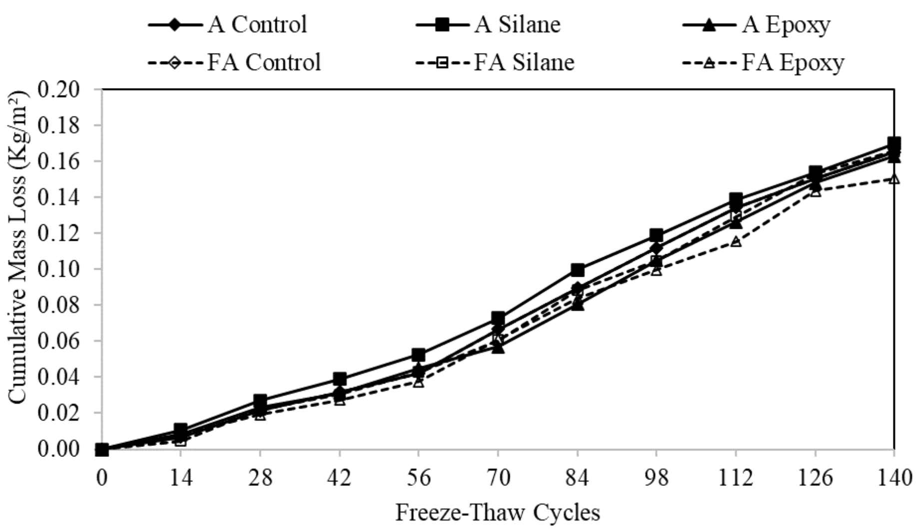


Figure 2.16 CDF-DI Cumulative Mass Loss from Scaling

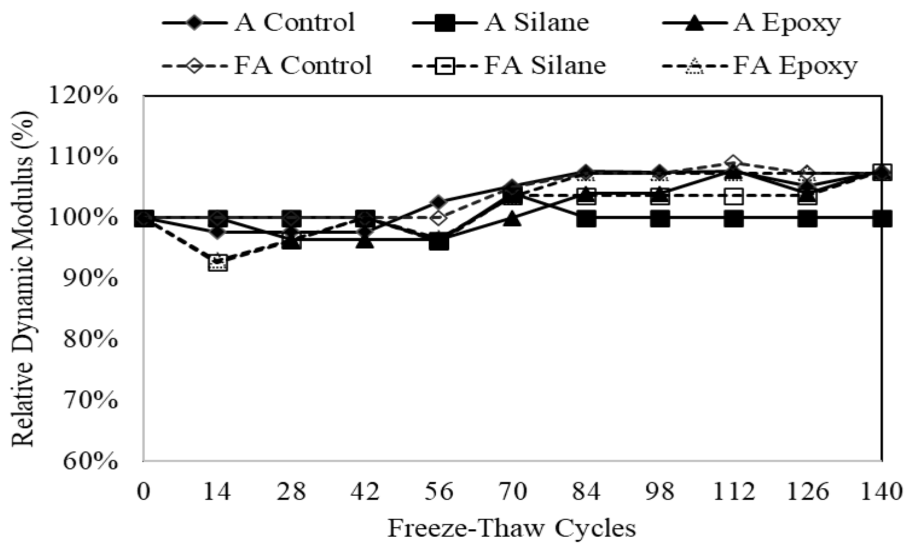


Figure 2.17 CDF-AI Relative Dynamic Modulus

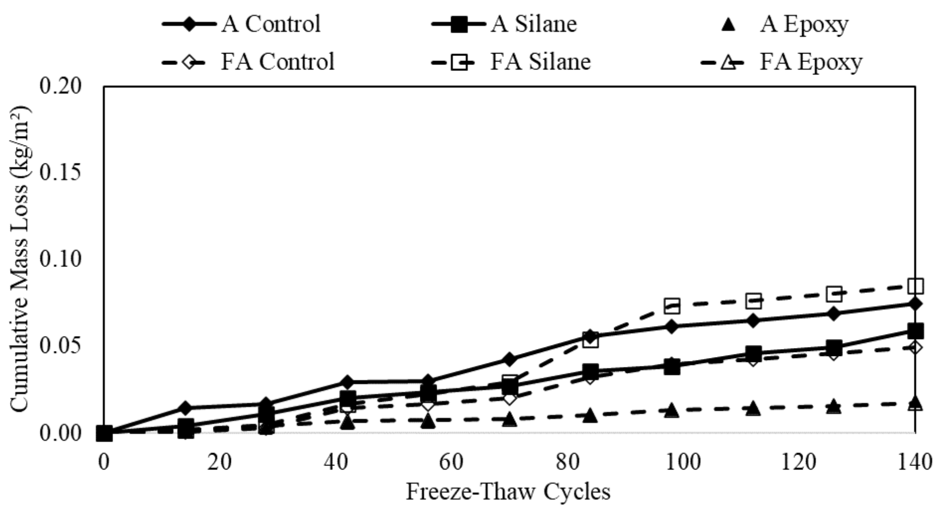


Figure 2.18 CDF-AI Cumulative Mass Loss from Scaling

After CDF testing chloride diffusion was determined through titration with the A sample results shown in Table 2.4 and the A-FA results in Table 2.5. The observed trends were as expected with the A-FA samples having lower chloride content than the A samples. In both cases the samples tested in the deicing fluid (CDF-DI) had higher chloride content than the liquid anti-icing brine condition (CDF-AI). The surface of the A specimens, in general, was rougher than the A-FA specimens which can be observed in Figure 2.8 due to the concrete setting more quickly during the field placement. This likely impacted application of the sealers and the cause of the silane and epoxy samples from the A mixture having comparatively higher chlorides than the A-FA mixture.

Table 2.4 Chloride Results from CDF Test Concrete A Specimens

Test	Concrete and Treatment	13mm	25mm
CDF-W	A Control	0.000%	0.000%
	A Silane	0.000%	0.000%
	A Epoxy	0.000%	0.000%
CDF-DI	A Control	0.247%	0.239%
	A Silane	0.240%	0.181%
	A Epoxy	0.101%	0.079%
CDF-AI	A Control	0.110%	0.071%
	A Silane	0.050%	0.045%
	A Epoxy	0.042%	0.025%

Table 2.5 Chloride Results from CDF Test Concrete A-FA Specimens

Test	Concrete and Treatment	13mm	25mm
CDF-W	FA Control	0.022%	0.020%
	FA Silane	0.016%	0.000%
	FA Epoxy	0.000%	0.000%
CDF-DI	FA Control	0.155%	0.103%
	FA Silane	0.027%	0.019%
	FA Epoxy	0.041%	0.028%
CDF-AI	FA Control	0.039%	0.030%
	FA Silane	0.030%	0.019%
	FA Epoxy	0.000%	0.000%

2.4.5 Simulated Tire Test Results

The simulated tire testing combined the anti-icer and deicer treatments with tire pressure for a year of representative storm events. Testing was only performed on A-FA concrete specimens and the results are shown in Table 2.6. The addition of the tire pressure and/or additional drying and wetting resulted in a significant increase to chloride penetration compared with the CDF samples which only experienced freezing and thawing and drying and wetting. The CDF specimens underwent 140 cycles of freezing and thawing and 10 cycles of drying and wetting while the simulated tire samples underwent 25 cycles of freezing and thawing and drying and wetting. It should be noted that the large increase in chloride content between the anti-icer conditions was due to the CDF-AI samples being tested in a larger volume of water during cycling than the snow applied to the simulated tire specimens. The results are comparison, not correlation. Similar to the previous results, the samples exposed to the liquid brine anti-icer had lower chlorides than the rock salt deicer condition. The sealer results were similar to the CDF testing with silane significantly reducing chloride ingress and epoxy providing the greatest reduction.

Table 2.6 Chloride Penetration from Simulated Tire Testing

Test	Concrete and Treatment	13mm	25mm
Water	FA Control	0.000%	0.000%
	FA Silane	0.000%	0.000%
	FA Epoxy	0.013%	0.004%
DI	FA Control	0.257%	0.163%
	FA Silane	0.137%	0.047%
	FA Epoxy	0.011%	0.000%
AI	FA Control	0.164%	0.158%
	FA Silane	0.072%	0.027%
	FA Epoxy	0.017%	0.008%

During testing the melt water was collected and chloride concentration measured for all conditions. For the size samples tested, the volume of runoff was similar for all samples. Figure 2.19 shows a comparison of the total chlorides applied during testing and the amount of chlorides collected in the runoff. As previously discussed and designed for in the test program, standard deicing practices using solid NaCl require more material per area than the liquid anti-icing brine protocol (i.e., 0.30 kg/m^2 DI applied vs. 0.20 kg/m^2 AI applied after 25 snow events in Figure 2.19). However, the amount of chlorides in the runoff are similar (i.e., 0.16 kg/m^2 DI runoff vs. 0.15 kg/m^2 AI runoff after 25 snow events in Figure 2.19). Therefore, more chloride retained in concrete in deicing applications than in anti-icing applications. This supports the findings presented previously where the deicing condition results in more chlorides in the concrete than the anti-icing condition.

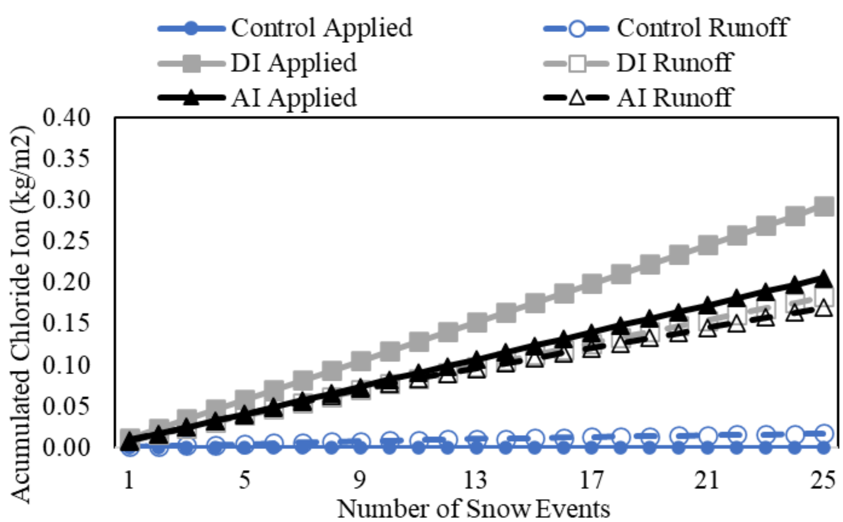


Figure 2.19 Applied Versus Runoff Chloride Content from Simulated Tire Testing

2.5 Summary

The objective of the laboratory study was to investigate the main difference between deicing and anti-icing (i.e., liquid chemicals are applied on dry concrete during anti-icing, while solid salt is applied on wet concrete covered with snow/ice during deicing) in controlled environment. All specimens used in the laboratory testing were casted on site using WIS 23 project concrete.

Laboratory testing was performed for A and A-FA samples either uncoated as a control or coated with silane or epoxy and tested in water, deicing, or anti-icing conditions. The specimens had good freeze-thaw performance indicating samples were collected from a high-quality and durable field mixture. In all cases the samples exposed to anti-icing treatment (i.e., 23% NaCl brine applied to dry concrete at 80 gallons/lane*mile) had better performance and lower chloride content than samples exposed to deicing treatments (i.e., solid NaCl at 250 lbs/lane*mile). The anti-icing samples had roughly half the amount of material scaled from the surface as the deicing samples. The chloride contents in anti-icing samples are also about half of the deicing samples. When tire pressure was used to simulate field conditions, chloride contents were up to 66% higher. The concentration of chlorides in the runoff water from the simulated tire testing samples was similar between solid material application as deicing and liquid application as anti-icing with the balance remaining in the concrete. Silane surface treatment provided a significant reduction in chloride penetration of around 50% for most conditions and epoxy effectively blocking chlorides with low to no chlorides penetrated. For these two concrete mixtures and for the NaCl conditions employed for this study, anti-icing should be the preferred winter maintenance practice.

Chapter 3 Accelerated Field Study at MnROAD

3.1 Introduction

It is known that the hydrodynamic pressure from traffic loading can increase the penetration of chloride into concrete pavement or bridge deck (Li Y., et al., 2022) (Yang, et al., 2022) (Li Z., et al., 2022). The objective of the field study was to investigate the combined effects of anti-icing chemicals, freeze-thaw cycles, and traffic loads.

3.2 Experiment Design and Construction

This task utilized the full-scale testing facility at MnROAD. MnROAD is a pavement test track made up of various research materials and pavements owned and operated by the Minnesota Department of Transportation. Located near Albertville, MN, temperature at MnROAD is very similar to central Wisconsin. Cell 37 on the Low Volume Road has reached its end of research life (Figure 3.1), so two slabs within this cell were took out and replaced with a concrete mix very similar to Wisconsin A-FA concrete. In addition, 12 concrete panels (2 ft * 2 ft size) casted in the WIS 23 Corridor Project (Fond du Lac to Plymouth) were placed into the fresh concrete. Figure 3.2 shows the layout of the 12 panels. Overall, the following three factors were included in this study:

- Bare concrete: Type A, Type A-FA.
- Mitigation techniques: penetrating sealer (silane), thin polymer overlay.
- Install location: inside wheelpath, between wheelpath.

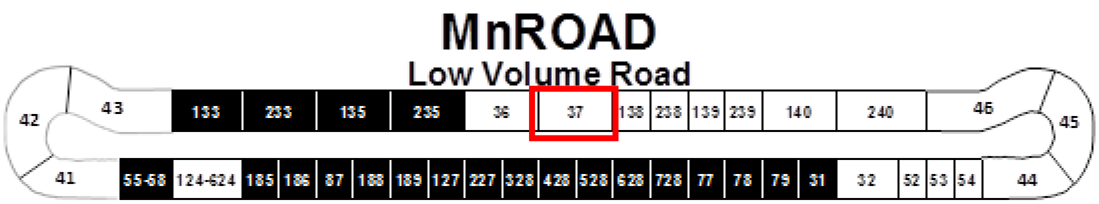


Figure 3.1 Location of Cell 37 on the low volume road at MnROAD

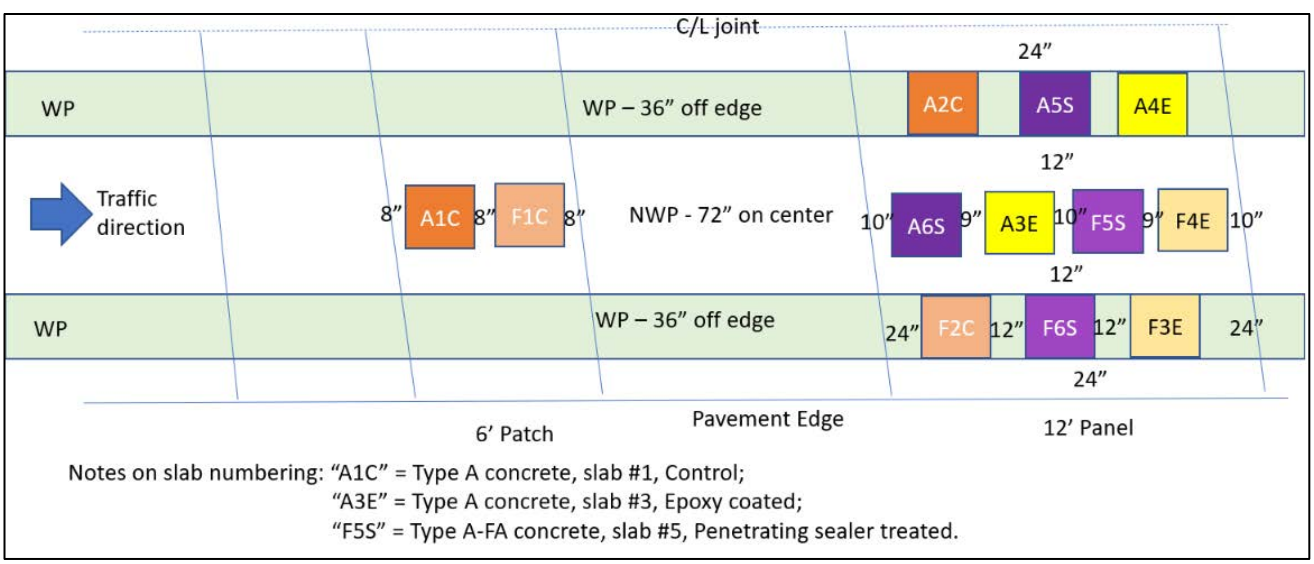


Figure 3.2 Placement of 12 concrete panels at MnROAD

During the construction on Nov. 2, 2021, the old slabs at MnROAD were first taken out, then rebars were set up so that the 2'*2'*2'' concrete panels can be placed in fresh concrete. It should be noted that the rebars were only intended to keep the panels from sinking into fresh concrete, not for reinforcement purpose. The concrete was covered with a tarp and cured for one month before anti-icing and traffic being applied. Figure 3.3 shows the construction process.

Salt brine (23% NaCl) was applied every week at 80 gal per lane mile during the winter seasons. A semi-truck with standard 80,000-pound load travels on the low volume road 80 laps per day, resulting in approximately 110,000 ESALs during the 17-month study period (December 1, 2021 to April 14, 2023).

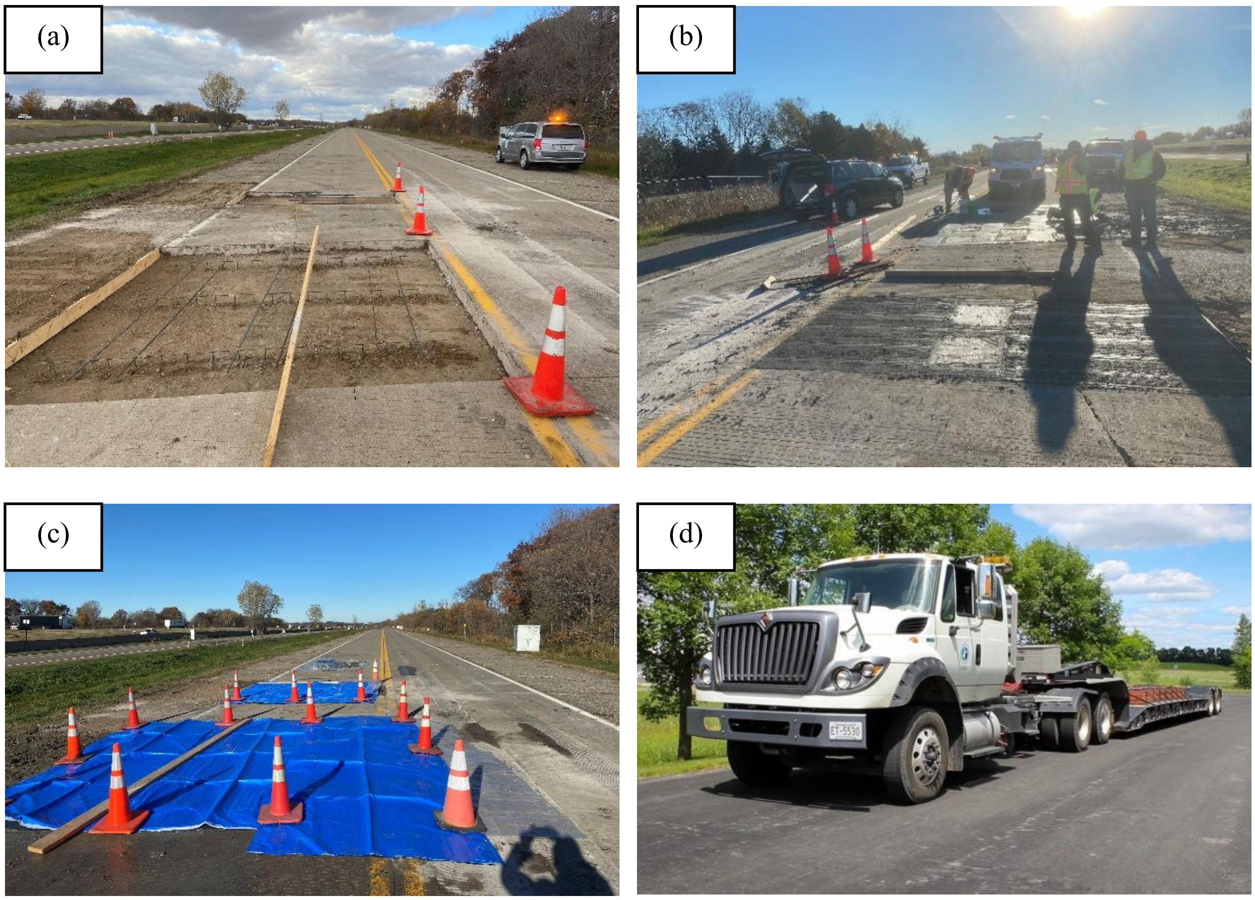


Figure 3.3 Construction process at MnROAD (a) old slabs being taken out and rebars set up for concrete panels, (b) construction completed on Nov. 2, 2021, (c) tarp coverage and curing for one month, (d) the 80,000-pound semi-trailer traveling around the closed-loop track at MnROAD

3.3 Field Performance

Figure 3.4 shows the performance during the 17-month study period. As expected, there was no visual distress on any of the panels due to the short time of study (i.e., the design life of concrete pavement is 20 years and the actual service life of concrete pavement is commonly 30~40 years).

Figure 3.4 Performance of concrete panels at MnROAD

Rebound hammer test (ASTM C805) was conducted during two site visits on 10/31/2022 and 4/14/2023. Using the conversion chart provided by the rebound hammer manufacturer, rebound numbers were converted to compressive strength. When compared with the compressive strength obtained from cylinder specimen (Figure 2.7), rebound hammer test showed lower strength for both Type A and Type A-FA concrete. This is contrary to the literature that "the rebound hammer test will record a higher in situ concrete strength because the rebound hammer test essentially tests the carbonated surface of the existing concrete structure" (Kog, 2018). This may be due to several reasons. For example, (1) the cylinder specimens were cured in lime-saturated water until testing, and (2) the in-situ concrete was only 1~2 years old so the surface was not likely carbonated. Since rebound hammer test was not calibrated with any laboratory specimens, the in-situ compressive strength is expected to have bias from laboratory-tested compressive strength.

Nevertheless, Figure 3.5 shows the following trends that agree with the literature:

1) WI Type A-FA concrete has 20~30% higher strength than WI Type A concrete. This agrees with the literature that fly ash increases the compressive strength.
2) WI Type A-FA concrete is 10% stronger than MN-new concrete. MN-new is also A-FA concrete designed to mimic the Wisconsin mix, but it was 1 year younger in age.

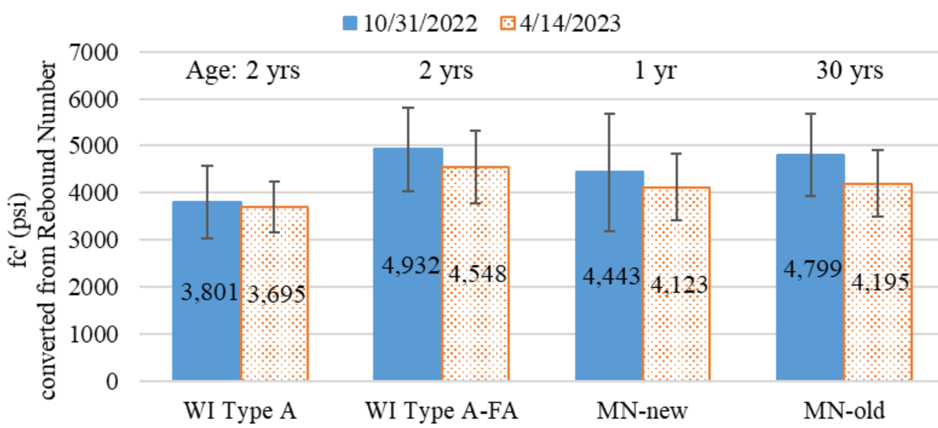


Figure 3.5 Compressive strength converted from rebound hammer test

During the two field visits, surface resistivity test was performed on in-situ concrete panels. Since the test was not on saturated specimen, the readings are for informational purposes only. Results are listed in Table 3.1 and a few observations can be made.

1) One year after the construction (10/31/2022), the epoxy and silane coating prevented the transmission of electric current resulting the meter reading "OF". This may be an indication of the protection from epoxy and silane. After another winter season (4/14/2023), readings were obtained on concrete with epoxy and silane coating, but readings were all higher than the control samples (without any coating). This may indicate the wear-off of the coating, as shown visually in Figure 3.4.

2) The average surface resistivity (4/14/2023) from high to low was epoxy (94), silane (89), and control (46). Again, this may be an indication of the effectiveness of surface protection.

3) The surface resistivity (4/14/2023) of type A-FA concrete is higher (47%, 3%, 21% for control, epoxy, and silane, respectively) than it for type A concrete. This is very likely attributed to the benefit of fly ash which reduces the permeability of concrete.

Table 3.1 Surface resistivity of concrete panels at MnROAD (unit: kΩ·cm)

Sample ID	Surface	Concrete	Location	10/31/2022		4/14/2023	
				Average	Std.Dev.	Average	Std.Dev.
A1C	Control	A	Center	82.0	18.6	32.1	1.4
A2C	Control	A	Wheelpath	145.3	31.5	42.0	12.2
F1C	Control	A-FA	Center	81.7	17.1	45.9	15.1
F2C	Control	A-FA	Wheelpath	133.7	6.8	63.0	16.7
A3E	Epoxy	A	Center	OF	N/A	79.0	15.6
A4E	Epoxy	A	Wheelpath	OF	N/A	117.0	N/A
F4E	Epoxy	A-FA	Center	OF	N/A	89.0	53.7
F3E	Epoxy	A-FA	Wheelpath	OF	N/A	112.0	N/A
A6S	Silane	A	Center	OF	N/A	88.8	52.1
A5S	Silane	A	Wheelpath	OF	N/A	80.5	19.8
F5S	Silane	A-FA	Center	OF	N/A	132.0	46.7
F6S	Silane	A-FA	Wheelpath	OF	N/A	73.0	22.6

Notes: "OF" – Overflow, the measured resistivity is out of range, typically >1000 kΩ·cm. "N/A" – Not available, because either no reading or only one reading was obtained.

3.4 Laboratory Test of Field Core Samples

To further evaluate the impact of anti-icing to the concrete panels at MnROAD, core samples were obtained on 4/14/2023 for laboratory testing. Chloride ion content was determined using acid titration according to ASTM C1152 and AASHTO T260. The test procedure and lab setup are described in section 2.3.3.

The chloride contents for type A concrete panels are shown in Table 3.2 and Table 3.3 for the "A-FA" concrete. In general, the resulting trends are consistent with laboratory findings. The A-FA control specimens had lower chlorides than the A specimens and the silane treatment produced approximately a 50% reduction in chloride ingress with epoxy providing the greatest reduction. The recovered specimens showed significant and visible wearing, both wheel path and non-wheel path, likely from plowing activities. The A-FA samples coated in epoxy did contain low but measurable chlorides, indicating the epoxy coating was breached. This could also explain Table 3.1 that surface resistivity was OF on 10/31/2022 but had readings on 4/14/2023.

Besides confirming the general trends observed in the laboratory, two significant observations are worth discussing.

1) The first significant observation is that in all cases the chloride content in the wheelpath was greater than the non-wheel path samples. When all data is pooled the results are statistically significant and the surface chloride concentration (1.6mm to 13mm) was 9.4% higher than the non-wheelpath samples and 40.9% higher at the lower depth (13mm-25mm). The comparison between the CDF-AI testing and the simulated tired testing also showed the combination of tire pressure and additional drying-wetting increased chloride ingress.

2) The second significant observation is the chloride content in the field specimens was less than the concentration observed from the simulated tire testing and more than the CDF-AI testing which did not include tire pressure. Although the field specimens had more applications of liquid brine anti-icing and more drying and wetting cycles, non-winter rainfall did remove some chlorides and reduced the diffusion gradient.

Table 3.2 MnROAD Sample Chloride Content for "A" Concrete

Concrete	Location	Treatment	13mm	25mm
A	Non-Wheel Path	Control	0.166%	0.103%
	Wheel Path	Control	0.172%	0.138%
A	Non-Wheel Path	Silane	0.062%	0.020%
	Wheel Path	Silane	0.080%	0.000%
A	Non-Wheel Path	Epoxy	0.000%	0.000%
	Wheel Path	Epoxy	0.000%	0.019%

Table 3.3 MnROAD Sample Chloride Content for "A-FA" Concrete

Concrete	Location	Treatment	13mm	25mm
A-FA	Non-Wheel Path	Control	0.091%	0.039%
	Wheel Path	Control	0.102%	0.072%
A-FA	Non-Wheel Path	Silane	0.043%	0.030%
	Wheel Path	Silane	0.033%	0.055%
A-FA	Non-Wheel Path	Epoxy	0.035%	0.018%
	Wheel Path	Epoxy	0.023%	0.017%

3.5 Summary

The objective of the field study was to investigate the combined effects of anti-icing chemicals, freeze-thaw cycles, and traffic loads.

Twelve concrete panels casted in Wisconsin were placed at Cell 37 of the Low Volume Road track at MnROAD. Factors included in the study were concrete type (A vs. A-FA), surface coating (control, sealer, epoxy), and install location (wheelpath, non-wheelpath). During the 17-month study period, salt brine (23% NaCl) was applied every week at 80 gal per lane mile during winter seasons, and a semi-truck with standard 80,000-pound load travels 80 laps per day, resulting approximately 110,000 ESALs.

There was no visual distress on any of the panels due to the short time of study. Based on rebound hammer test, it was found that WI Type A-FA concrete had 20~30% higher strength than WI Type A concrete, proving the benefits of fly ash. Surface resistivity testing provided an indication of the effectiveness of surface protection (i.e., concrete coated with epoxy and silane had higher surface resistivity than bare concrete). In addition, wearing-off was observed on epoxy and silane coated panels after two winter seasons.

Core samples were taken from the field for laboratory testing. Based on chloride content, the A-FA concrete had lower chlorides than the A concrete, again proving the benefits of fly ash. The silane treatment produced approximately a 50% reduction in chloride ingress with epoxy providing the greatest reduction.

In terms of the effect from traffic, data showed that in all cases the chloride content in the wheelpath was greater (9% at surface and 41% at the lower depth) than the non-wheel path samples, confirming with the hypothesis that tire pressure greatly increases the ingress of chloride to concrete.

Chapter 4 Analysis of Pavement and Bridge Management Data

4.1 Introduction

Wisconsin has increased the application of anti-icing in the past decade. If anti-icing were to create more damage to concrete, this change of winter maintenance practice may have led to observable performance change in the field. With this premise, the research team analyzed historical performance data of Wisconsin pavements and bridges, as well as winter maintenance record from WisDOT.

For pavements, the main distresses examined included joint spalling, scaling & map cracking, and popouts. Data in the pavement management system showed very few distresses. Regardless of when pavements were surfaced, the results of the analysis did not indicate any definitive relationship between the observed distress density and salt application rate. Details of the analysis are included in Appendix A.

For bridges, since maintenance record was available, data analysis led to more conclusive results pertinent to the objective of this study. The following sections explain the process to integrate the bridge management data and winter maintenance data, and to investigate any correlations between bridge performance and salt application (solid salt, salt brine, and quantities). The hypothesis was that more salt application could have caused more damage and hence more frequent maintenance to bridges. The effect of coating with penetrating sealer or epoxy was also included in the analysis.

4.2 Data Collection for Bridge Condition and Salt Usage

Bridge information and performance were from Highway Structure Information System (HSIS) database provided by WisDOT. The database contained bridge location, traffic data, bridge configurations, deck geometry, construction and maintenance history and costs, inspection history, and National Bridge Inventory (NBI) component condition ratings.

The total recordings of documented bridge data were 26,372 sets with construction years from 1880 to 2021. These bridges could be categorized based on main span materials such as concrete, steel, timber, and others. The records of different bridges are listed in Table 4.1.

Table 4.1. Record number of bridges with different main span materials

Main span materials		Total number of bridges
Concrete	Reinforced Concrete	8,928
	Continuous reinforced concrete	5,354
	Continuous prestressed concrete	1,956
	Prestressed Concrete	2,760
Steel	Steel	3,512
	Continuous jointed steel	1,659
	Galvanized steel	977
Timber		766
Others		463

The wearing surface of bridge deck may not match the main span material or deck surface as recorded since it varies by different types of overlays and surface treatments. To investigate deck performance of concrete based bridges, those with main span material of concrete were extracted and further filtered based on typical types of wearing surface, including concrete, integral concrete, low slump concrete, epoxy overlay concrete, polyester polymer concrete, micro silica modified concrete, latex concrete, and the ones without surface treatment. The filtered bridge data was organized and filtered to eliminate unreasonable data. After filtering, the bridge database has a total of 8,412 unique bridges with typical bridge information and annual recordings of deck maintenance & preservation activities and deck condition rate.

Figure 4.1 shows the distribution of construction year of all recorded bridges and the application time of major deck maintenance activities. As it can be seen from Figure 4.1, most of bridges were constructed within the past 70 years. The average bridge ages at deck overlay and replacement are around 25 and 50 years, respectively.

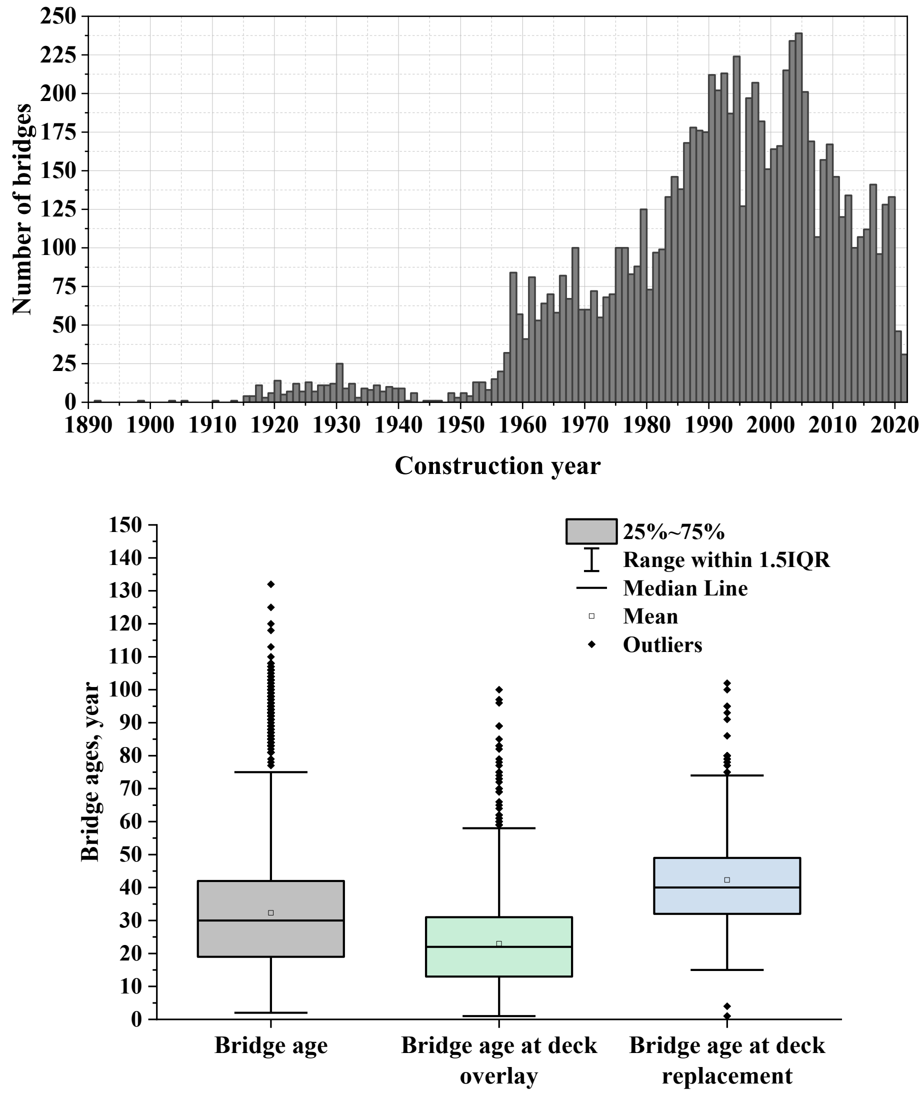
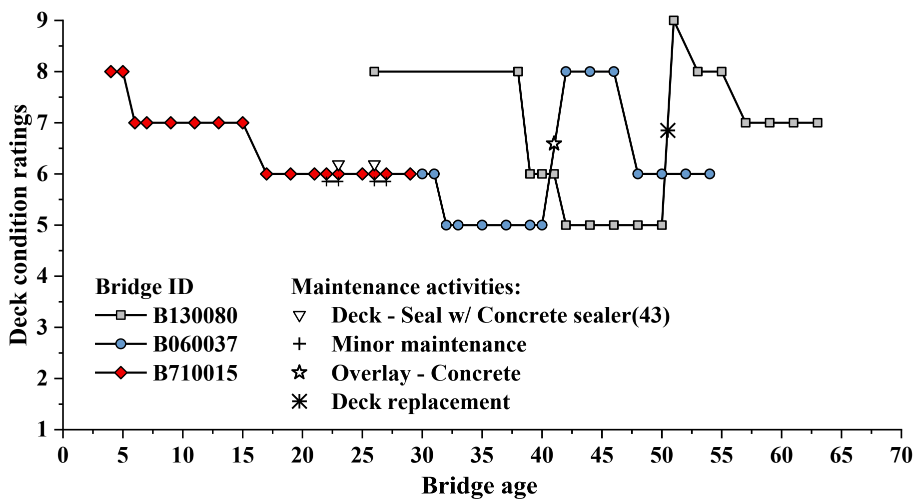


Figure 4.1 (top) Concstruction years of recorded bridges, (bottom) application time of major deck maintenance.

The bridge deck condition is evaluated by NBI condition rating method with a range from 0 (failed condition) to 9 (excellent). Based on the bridge database, the annual deck condition ratings were combined with construction and maintenance historical data. The effect of maintenance activities on annual deck conditions were analyzed. An example is shown in Figure 4.2. The three different bridges are located in northwest, northcentral, and southwest of Wisconsin. Red dots show that bridge B710015 started with condition 8, then dropped to 7, then 6. Concrete sealer and minor maintenance were then applied to this bridge, but the deck condition rating stayed at 6. For bridge B060037 (blue dots), an overlay was applied at age 42 and increased the deck rating from 5 to 8. For the gray dots (B130080), deck replacement was applied at age 51 and increased the deck rating from 5 to 9.



Bridge ID	County	Feature Over	Highway	Latitude	Longitude
B060037	Buffalo	Harvey Creek	STH 37	44.5576778	-91.6885278
B710015	Wood	Yellow River branch	STH 80	44.3503583	-90.1113000
B130080	Dane	Maunesha River	USH 151 SB	43.2343000	-89.1536833

Figure 4.2 Effects of maintenance activities on bridge deck condition ratings.

The bridge deck ratings before and after maintenance activities were summarized in boxplots, as shown in Figure 4.3. The influences of deck replacement and overlay on deck rating were both significant. Deck replacement can efficiently recover deck condition from average rating of 5.3 to 8.5; while overlay can improve deck condition from average rating of 6.3 to 7.1. For crack sealing, deck repair & patching, no apparent improvement in deck condition ratings were noticed.

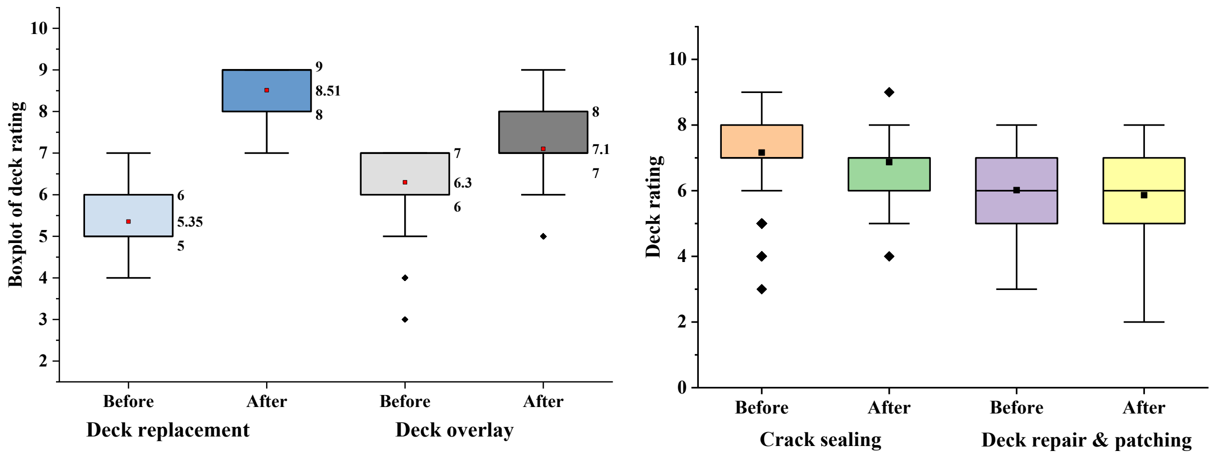
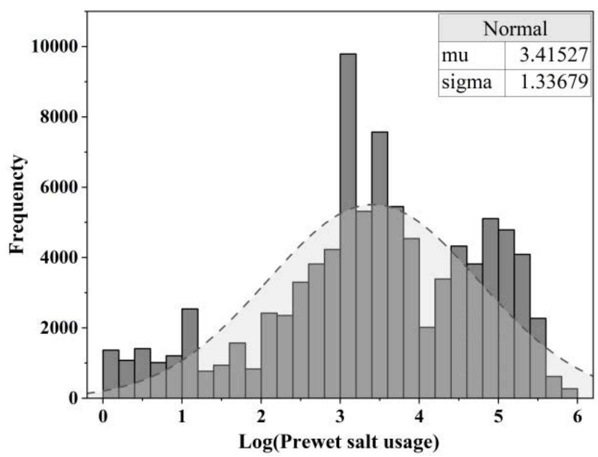
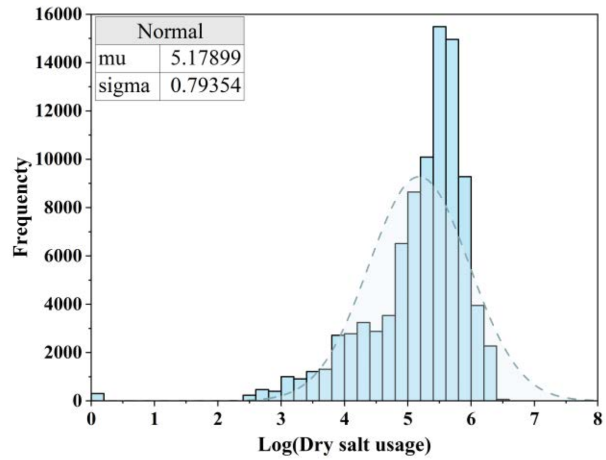


Figure 4.3 Deck condition ratings before and after maintenance activities.

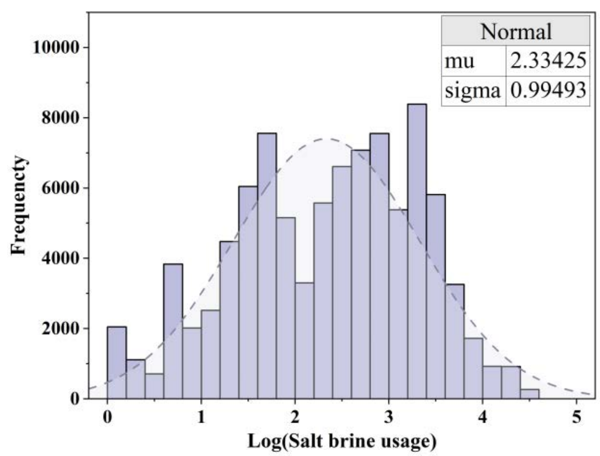
Annual salt usage data were obtained from winter maintenance program provided by WisDOT. The salt usages include three types of salt: prewet salt, dry salt, and salt brine. Figure 4.4 shows the distributions of annual average salt usages (pounds per year) at different route segments in Wisconsin. The results show that the salt usage has variations across different locations. More details about salt usage in Wisconsin can be found in WHRP 17-03 final report (Xiao, Owusu-Ababio, and Schmitt, 2018).



(a)



(b)



(c)

Figure 4.4 Distributions of annual average salt usages for (a) prewet salt; (b) dry salt; and (c) salt brine (pounds per year)

4.3 Machine Learning Model for Predicting Deck Condition

Machine learning is a type of artificial intelligence technology that allows data scientists to develop computerized programs to accurately predict outcomes without using traditional manual statistical methods. The prediction of machine learning is realized relying on typical algorithms to generate new output values based on input datasets. Random Forest is a popular machine learning algorithm that produces the output by combining the results from multiple randomly selected decision trees.

4.3.1 Random Forest Algorithm

As shown in Figure 4.5, the development of random forest machine learning model is started by splitting the main database into training dataset (25%) and testing dataset (75%). The training dataset is used for generating decision trees to build the random forest model with numerous decision trees. All decision trees are generated based on Bootstrap theory, by which the datasets and variables used in each decision tree are randomly selected from the training dataset. Repeated selection is allowed. Each node in decision trees is split based on Gini impurity, which is a factor representing the impurity of dataset at each node. The variable with the lowest Gini impurity is used as the top root node. The Gini impurity is calculated as Equation (1).

$$Gini\ impurity = 1 - \sum\nolimits_{i=1}^{n} {p_i}^2 \qquad (1)$$

where, p_i is the percentage of data number with the same response at that node.

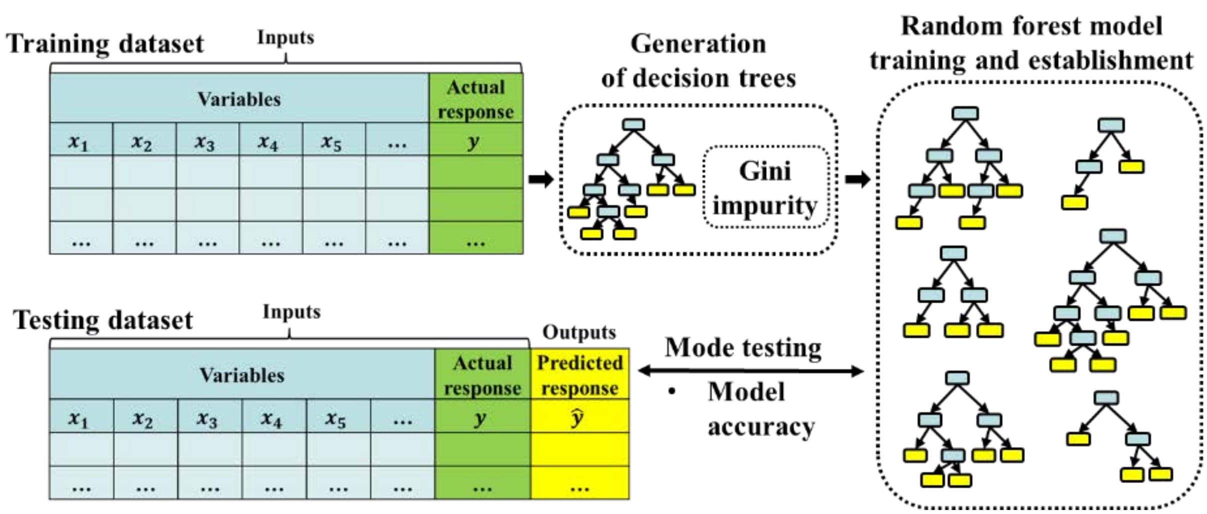


Figure 4.5. Random forest machine learning model

After the model training and establishment, the variable importance can be traced to present the relative influence of each variable in the training dataset. It is calculated based on Equation (2).

$$I_i = \frac{1}{F}\sum_{j=1}^{F}\sum\nolimits_{k=1}^{N} p_k \Delta_{gini} \qquad (2)$$

where, I_i is the importance of variable x_i; F is the total number of decision trees in the random forest model; N is the number of nodes using variable x_i; p_k is the percentage of data number with the same response at that node; Δ_{gini} is the change of Gini impurity between that node and the nodes in sub layer.

The testing dataset is used to verify the model efficiency, presenting the prediction accuracy. The model prediction accuracy is calculated as Equation (3):

$$Accuracy = \frac{1}{N}\sum_{i=1}^{N} \frac{|y_i - \hat{y}_i|}{y_i} \quad (3)$$

where, N is number of testing datasets; y_i and $\hat{y}_i$ are actual and predicted responses, respectively.

4.3.2 Development and verification of Random Forest model

The Random Forest algorithm was used to predict bridge deck ratings. The input parameters included bridge/deck age, application time of different preservation and maintenance activities, average daily traffic (ADT) data, annual average salt usage, and general bridge attributes (such as length, deck lane, deck materials).

Considering that major maintenance activities such as overlay and deck replacement have dominant influences on deck condition, the entire service life of bridge deck was classified into four stages, as shown in Figure 4.6. The four stages include the following:

1) Stage 1 is the period from initial construction to the first overlay activity.
2) Stage 2 is the period from deck overlay to the first deck replacement.
3) Stage 3 is the period from the first deck replacement to the next overlay.
4) Stage 4 covers the rest of period until the end of service life that may contain multiple overlay activities.

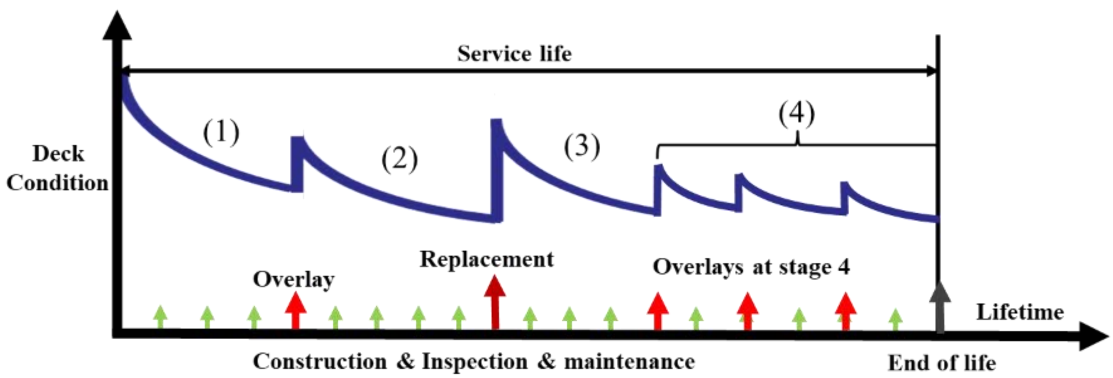


Figure 4.6 Bridge deck service life prediction at different stages

Based on the classified stages, four Random Forest machine learning models were developed separately at each stage. The input variable of bridge and deck age were time factors that respectively indicate the age of bridge and deck since initial construction or deck replacement. The overlay age indicated the years since the last overlay was applied. The application of concrete sealer (silane/siloxane) was input as binary index (0/1). The average daily traffic values were input as ADT of car and truck, respectively. The salt usages were input based on different salt types (prewet salt, dry salt, and salt brine). Other inputs of bridge deck attributes included deck length, main span length, overburden depth, main span materials, deck lane number, and so on.

The general training results of RF models are listed in Table 4.2. The performance factors include mean absolute error (MAE), R-square, root mean square error (RMSE), and accuracy as shown in Equation (3). According to the training results, all four RF machine learning models show good accuracy for predicting bridge deck ratings with R-square values of 0.72-0.93.

Table 4.2 Testing results of RF machine learning models

Stages	MAE	R-square	RMSE	Accuracy	Data number
1	0.40	0.72	0.30	94.25	74,410
2	0.38	0.74	0.33	93.69	13,012
3	0.27	0.81	0.16	96.04	4,717
4	0.18	0.93	0.06	97.15	152

Figure 4.7 shows importance of input variabels from RF models. Only variables with importance values greater than 0.05 are presented. As it can be noticed that bridge age and deck age show relatively higher importance compared to other input variables. Overlay age shows significant importance only at stage 2 and 4. The average daily traffic of car (ADT car) presents the third importance rank in stage 1, 3, and 4, and shows the secondary importance in stage 2. The average daily traffic of truck (ADT truck) shows relative low importance at all stages. The importance of three types of salt usages are similar. As for other bridge deck features, deck length and main span length show higher importance among these deck features except stage 4, in which the overburden depth has higher improtance. It is noted that the importance of concrete sealer is low (<0.05), but it is included in the input variables for further analysis.

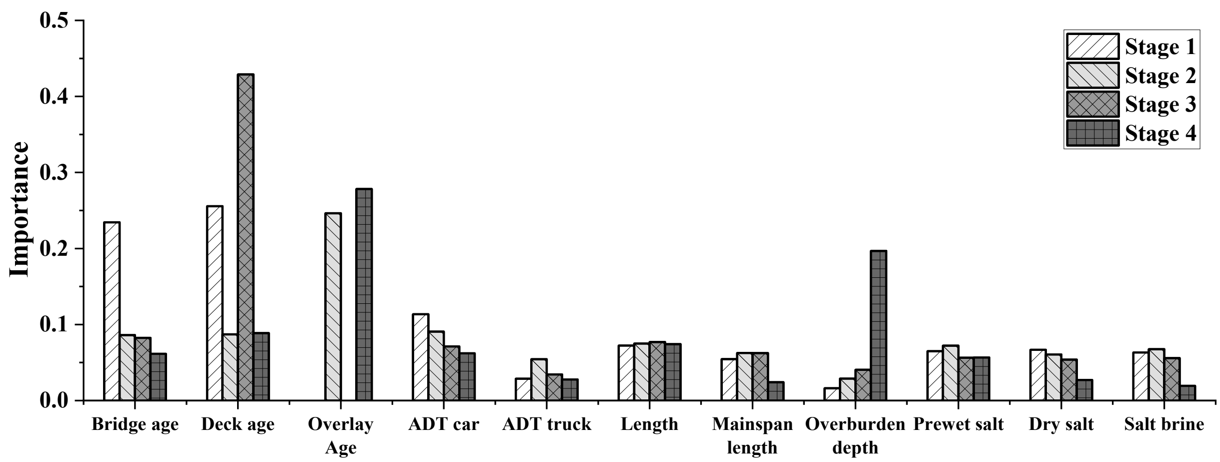


Figure 4.7 Importance of input variables for bridge condition prediction

Figure 4.8 presents two examples of predicted deck ratings for 100-year service life of bridge using the inputs of two recorded bridges in the database. The prediction model shows good capability to predict the variations of deck condition that match well with the actual data. The effects of overlay and replacement on deck condition ratings can be clearly observed.

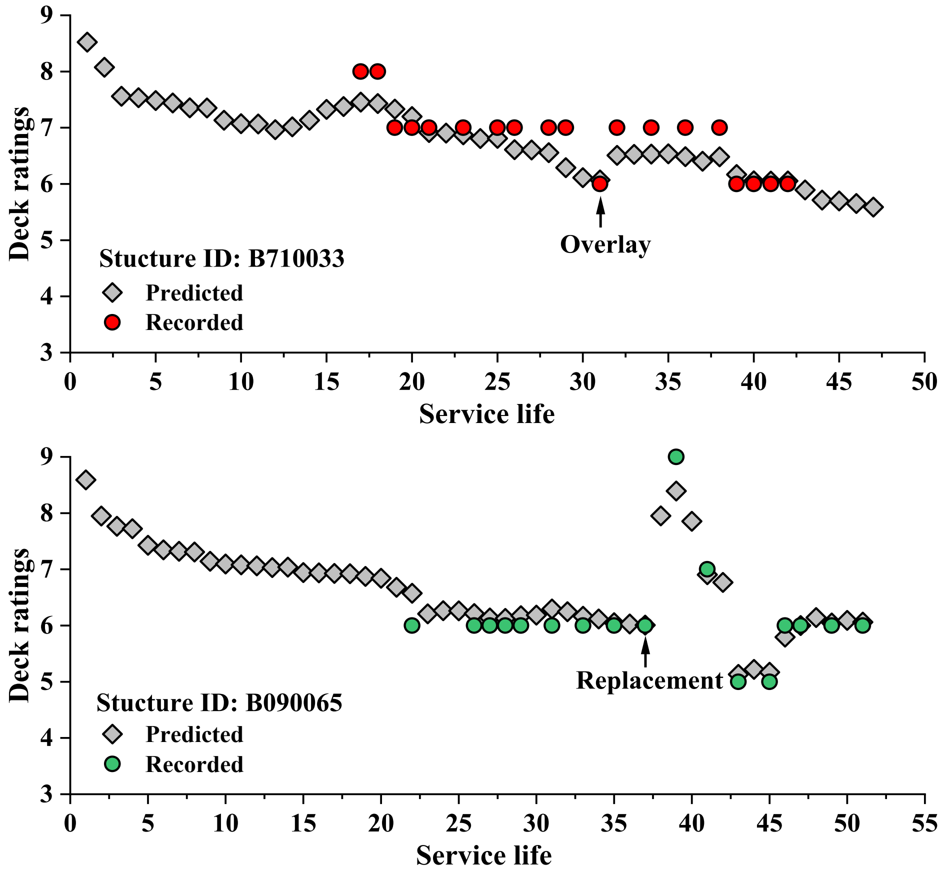


Figure 4.8. Predicted deck ratings for two selected bridges

4.3.3 Prediction of bridge deck condition

To investigate the effects of certain input variables on deck condition ratings, further analysis was conducted based on the developed RF machine learning models. Table 4.3 presents the design of two prediction scenarios. For each scenario, bridge attributes such as lane number, length, deck materials, etc., were randomly assigned based on the database and input into the model for prediction. The effects of concrete sealer and salt usage on deck condition ratings were analyzed as follows: 1) for concrete sealer, the comparison was between bridges with and without sealer applications; 2) for salt usage, the salt effect was represented by comparing predicted deck ratings of bridges with less, moderate, and more salt usages (including three types of salts).

Based on the analysis of bridge deck ratings in the database, the maintenance schedule was determined based on the rule that deck overlay was applied when the rating was below 6.3 and deck replacement was triggered when the rating was below 5.2. It was noted that one overlay was applied before the first deck replacement, while multiple overlays might be applied after replacement during the 100-year period.

Table 4.3 Prediction scenarios of bridge deck conditions

<table>
<tr><th>Prediction scenario</th><th>Investigated factor</th><th>Results comparison</th><th>Prediction setup</th></tr>
<tr><td>1</td><td>Concrete sealer effect</td><td>No concrete sealer vs. concrete sealer</td><td rowspan="2">• Predicted service life: 100 years
• Input data: bridges with random attributes based on database.
• Preservation & maintenance schedule was analyzed based on the criteria below.
o Overlay is applied when deck rate is below 6.3.
o Replacement is triggered when deck rate is below 5.2.
o One overlay applied before replacement and multiple overlays may be applied after replacement.</td></tr>
<tr><td>2</td><td>Salt effect (Prewet salt, dry salt and salt brine)</td><td>Less vs. moderate vs. more salt (25th percentile vs. average vs. 75th percentile)</td></tr>
</table>

Figure 4.9 shows the prediction results of three bridges at each case based on the inputs in Table 4.3. For the first bridge in Figure 4.9, the effect of concrete sealer on bridge deck condition is observable at stage 1. Although the predicted deck condition ratings with and without concrete sealer show different trends at the beginning of stage 2, the deck replacement is triggered at almost the same year. After deck replacement which recovered the condition rating to around 8.0, the bridge with concrete sealer shows slightly higher rating. For the other two bridges in Figure 4.9, the predicted deck condition ratings with concrete sealer are slightly higher than that of without concrete sealer at stage 3 and 4. The results indicate that applying concrete sealer may better preserve deck ratings, but the effect is not significant.

The effects of different types of salt usages on predicted deck ratings are shown in Figure 4.10. According to the predicted deck condition ratings, only minor differences could be observed for bridge decks with different salt usages. More usages of prewet salt and salt brine result in slightly higher deck condition ratings, while less dry salt usage presents slightly higher deck condition ratings. Although the difference is not dramatic, this finding agrees with laboratory study as discussed in Chapter 2 that anti-icing (with salt brine) created less damage than deicing (with dry salt).

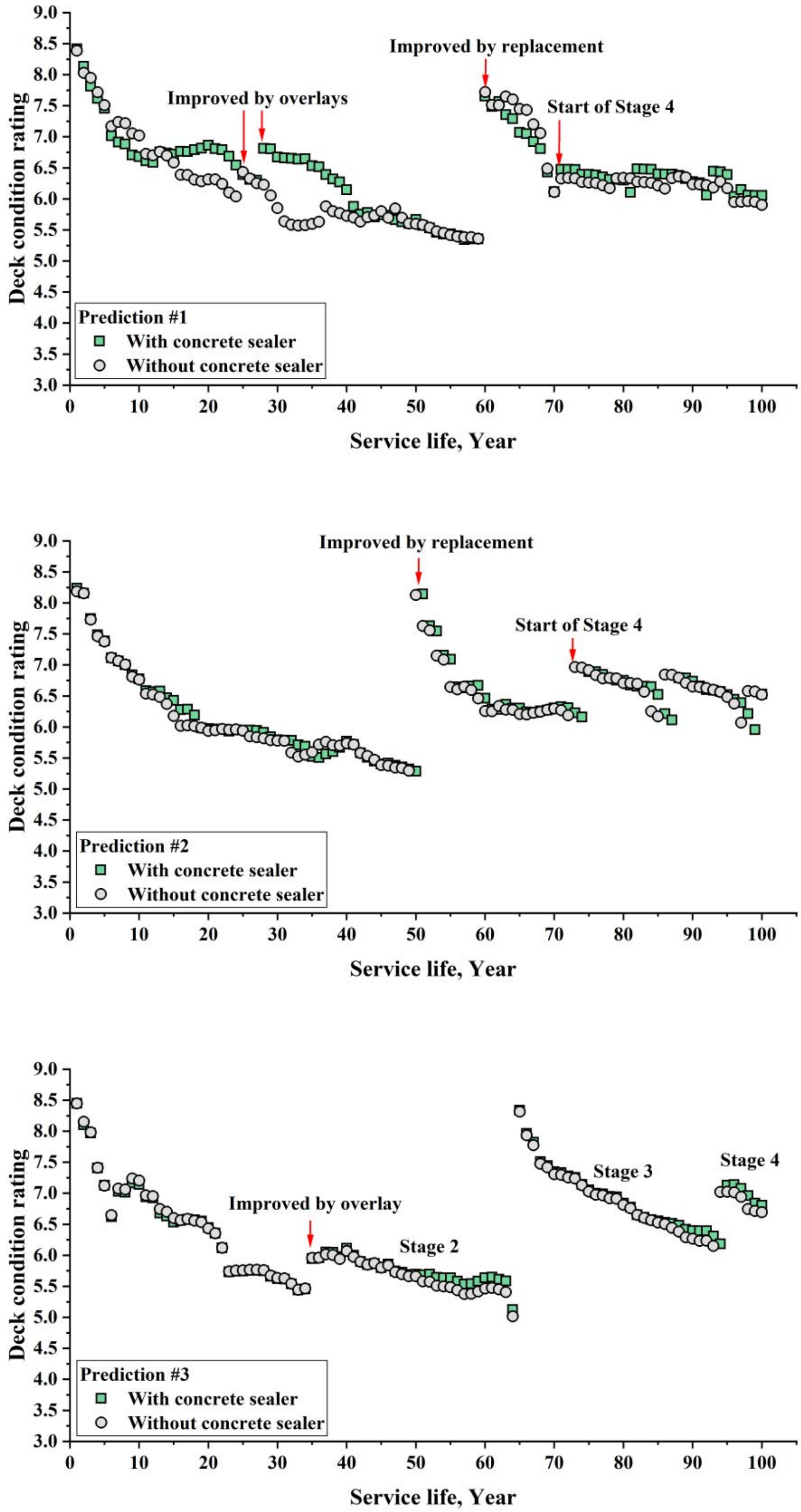
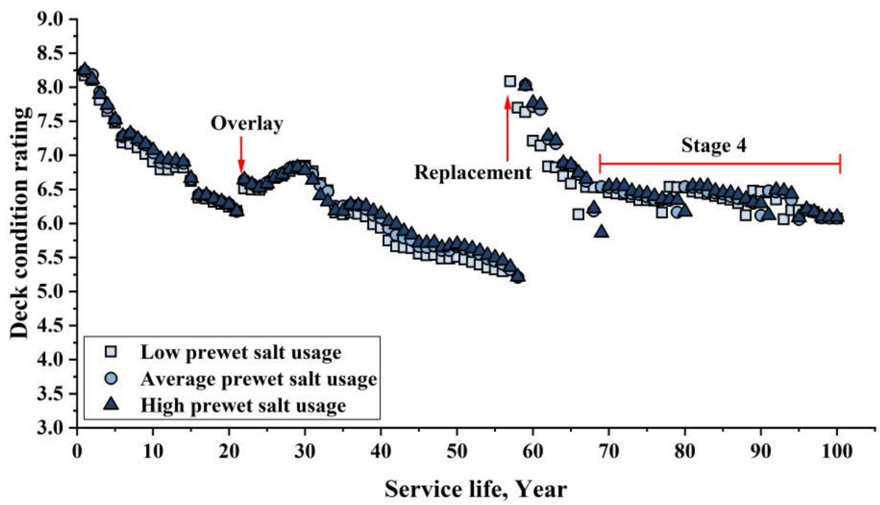
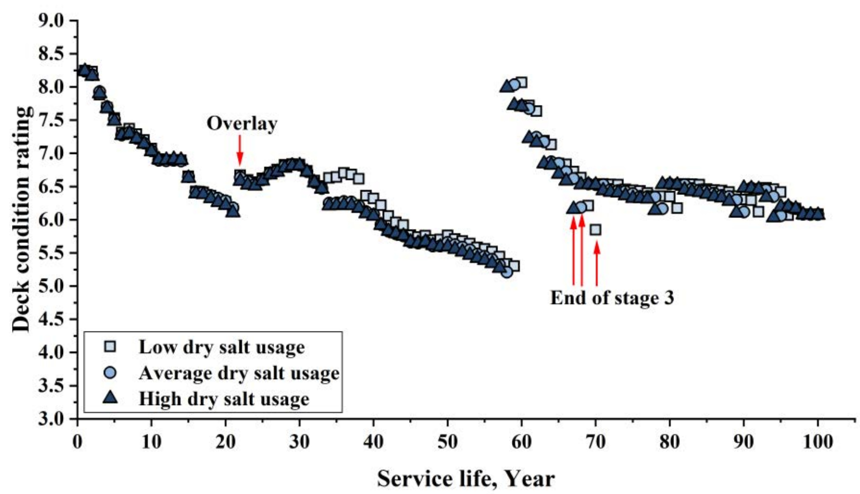


Figure 4.9 Effect of concrete sealer on deck condition ratings.





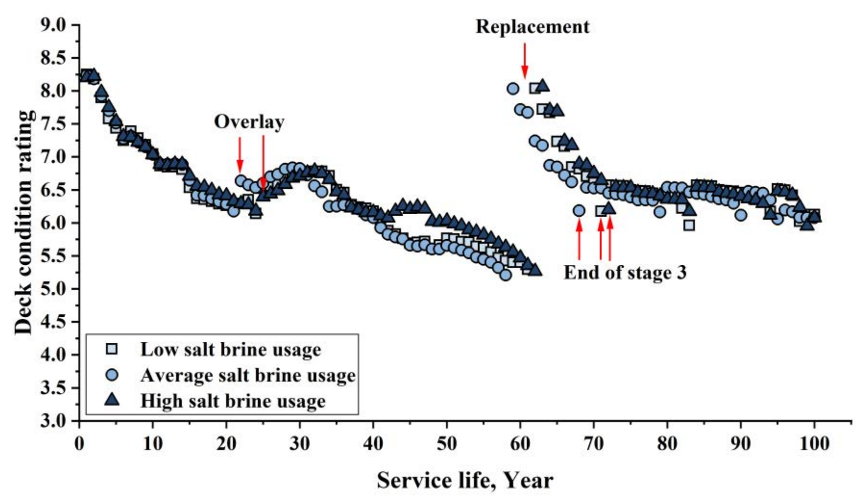


Figure 4.10 Effect of salt usage on deck condition ratings.

4.4 Life Cycle Cost Analysis

4.4.1 Orthogonal analysis of factors affecting bridge deck rating

Previous analyses found that the influence of concrete sealer and salt usage on bridge deck condition ratings are not significant and varied a lot among different bridges. Thus, orthogonal analysis was conducted to further quantify the influence of concrete sealer and salt usage. Orthogonal analysis is a mathematical method that is usually used for investigating the effects of multiple factors with levels in a simplified way. For instance, to fully conduct experiments with 4 factors and 2 levels for each factor, it requires 2^4 = 16 tests. However, the tests can be reduced to 8 tests with an orthogonal design table.

Table 4.4 shows an orthogonal design table listing prediction inputs for 4 factors and 2 levels. The four factors are concrete sealer, amount of prewet salt, amount of dry salt, and amount of salt brine. The bridge deck with and without concrete sealer treatments are denoted by level 1 and 2, respectively. The salt usages at 25th and 75th percentile of the values as recorded in the database are set, respectively, as level 1 and 2. For each case, the prediction was conducted for 100 years and multiple overlays were allowed at stage 4 to maintain the deck rating above 6.3. The maintenance schedule is determined based on the rule that deck overlay is applied when the rating is below 6.3 and deck replacement is triggered when the rating is below 5.2.

Table 4.4. Orthogonal table of prediction model inputs

Factor	Sealer	Prewet Salt (lb)	Dry Salt (lb)	Salt brine (lb)
1-1	1 (Yes)	1 (562)	1 (43,146)	1 (99)
1-2	1	2 (44,150)	2 (224,420)	2 (1,706)
1-3	1	1	2	2
1-4	1	2	1	1
1-5	2 (No)	2	1	2
1-6	2	1	2	1
1-7	2	2	2	1
1-8	2	1	1	2

The prediction results of 8 cases are plotted in Figure 4.11 to compare the service life at stages 1-3 and the number of overlays at stage 4. The concrete sealer is applied at every 2 or 4 years and the results are similar. The results are averaged to show the effect of level at each factor. As for concrete sealer, the bridge decks with concrete sealer treatment show longer service life at stages 1-3 and requires fewer overlays at stage 4. The analysis results indicate that applying concrete sealers can help extend service life of bridge deck by around 1~2 years at stage 1-3 and reduce the number of overlays at stage 4.

As for salt usage, dry salt presents insignificant influence on service life of stage 1, while less dry salt can help extend the service life of stage 2 and 3 by around half a year and reduce one overlay at stage 4. As for the other two types of salts, more usage of these salts shows positive effect on extending service life by around 1~2 years and reducing one overlay at stage 4. Since the effect of salt usage on bridge deck condition ratings are not significant and the cost data of deicing salts on each bridge is unavailable, only the effect of concrete sealer is considered in life cycle cost analysis.

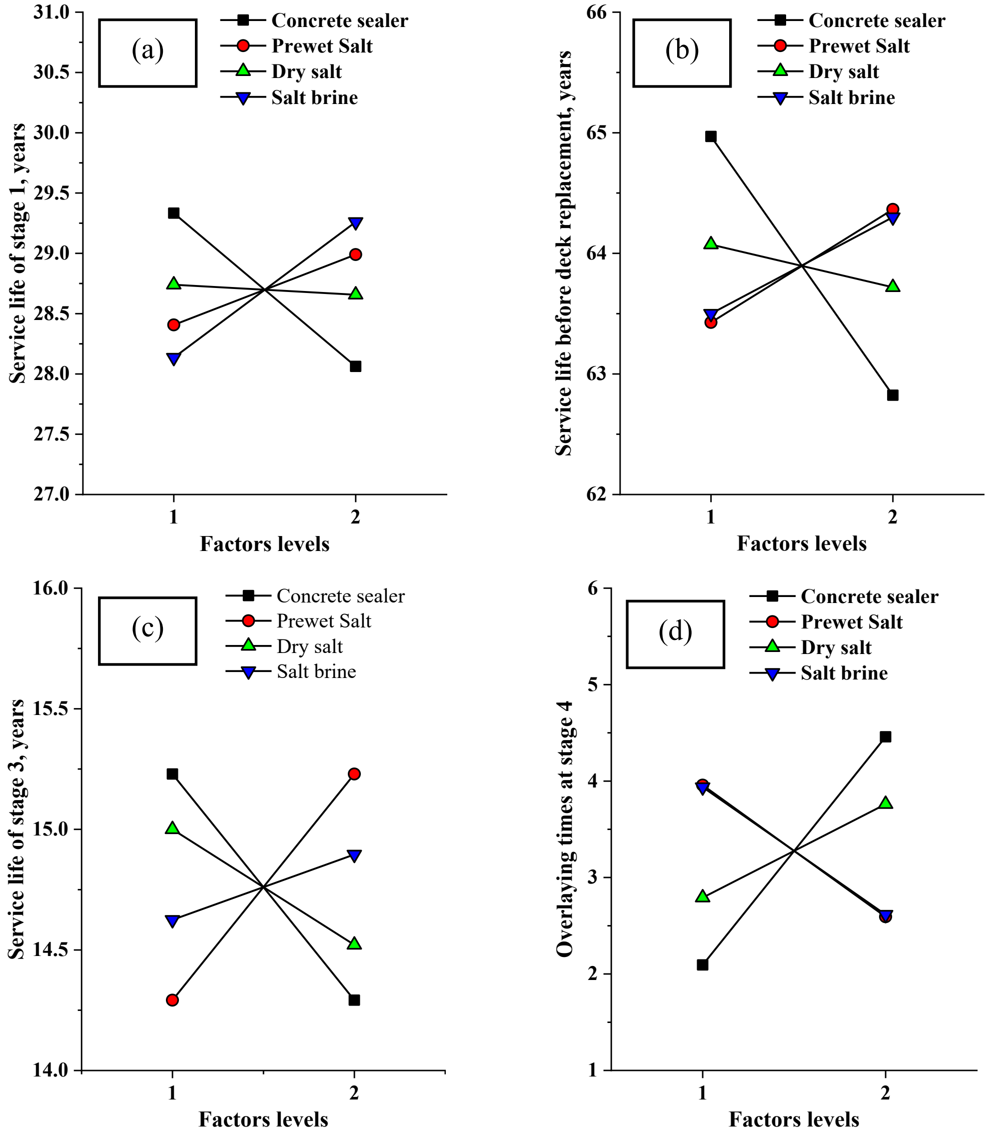


Figure 4.11. Effects of factor levels on service life at (a) stage 1, (b) stage 1 + stage 2, (c) stage 3, and (d) number of overlays at stage 4.

4.4.2 Life cycle cost analysis

Life cycle cost analysis (LCCA) is a systematic tool used to assess the total cost of owning a facility or running a project with flexibility and comprehensiveness. LCCA can be used to calculate all significant and relevant costs over the total life cycle of bridge. LCCA is usefully applied to determine bridge design alternative that will fulfill the project objective at the lowest overall cost while with satisfied service level and performance.

The total cost of a bridge deck during its service life is mainly constituted of initial construction cost, routine inspection costs, maintenance costs, demolition cost and residual value, which can be expressed as Equation (4) (Federal Highway Administration, 2002).

$$LCC_{NPV} = C_{ic} + \sum_{i=1}^{n_{ri}} \frac{C_{ri} t_i}{(1+r)^{t_i}} + \sum_{k=1}^{n_{mt}} \frac{C_{mt} t_k}{(1+r)^{t_k}} + \frac{C_d}{(1+r)^T} - \frac{R_v}{(1+r)^T} \tag{4}$$

where, LCC_{NPV} is the total cost represented by Net Present Value (NPV); r is the monetary discount rate; C_{ic}, C_{ri}, C_{mt}, C_d and R_v are costs of different activities: initial construction, routine inspection, maintenance, demolition and residual value, respectively; n_{ri} and n_{mt} are number of corresponding activities during the investigated period; T is the investigated service life.

Based on the NPV, the equivalent annual cost can be calculated as an indicator for comparison of costs of bridge decks with different life periods. The equivalent annual cost is calculated as shown in Equation (5).

$$EUAC = LCC_{NPV} \frac{r}{1-(1+r)^{-T}} \tag{5}$$

where, $EUAC$ is equivalent uniform annual cost, it equals to $LCCA_{NPV}/T$ when the monetary discount rate is zero.

Table 4.5 summarizes the cost data of different maintenance activities of concrete bridge decks used in LCCA. The costs of new deck, deck replacement and overlay are extracted from Bureau of Structures Cost Estimate Calculations of WisDOT (Wisconsin Department of Transportation, 2023). The new deck cost and deck replacement cost are regarded as the same, \$105/SF for concrete bridge deck. The deck overlay cost is \$46/SF.

Typical minor maintenance methods with corresponding costs are extracted and calculated from the bridge database, which are \$1.9/SF for deck repair and \$1.4/SF for crack sealing. The unit cost of concrete sealer is obtained from the communication with WisDOT, which is \$0.7/SF. All minor maintenance and concrete sealer costs are unit costs based on the total area of deck surface, in keeping the same unit with deck replacement and overlay in LCCA. The cost of routine inspection is \$0.2/SF based on the data in the literature (Cusson, Lounis, & Daigle, 2010).

Table 4.5. Cost data of maintenance activities for bridge decks

Subjects		Cost, $/m^2	Cost, $/SF
Major maintenance	New deck/Deck replacement	1130	105.0
	Deck overlay	495	46.0
Minor maintenance	Deck repair	20	1.9
	Crack sealing	15	1.4
Concrete sealer		8	0.7
Routine inspection		2	0.2

Table 4.6 lists the application intervals of deck preservation based on literatures and observations in the bridge database. The routine inspection and deck clean, sweep and drains are regularly applied every 2 years. Typical minor maintenance such as crack sealing, deck repair and patching are applied every 2-6 years. The application interval of minor maintenance is set as 4 years in LCCA.

Table 4.6. Application intervals of deck preservation

Preservation activities	Application intervals (Years)
Routine inspection	2
Deck clean, sweep and drains	2
Deck crack sealing	2~6
Deck repair & patching	2~6

Figure 4.12 compares the predicted major maintenance schedules for 12 different bridge decks based on the developed performance prediction model, respectively, with and without concrete sealer applied every 2 years. The major maintenance activities include deck overlays and deck replacement. It is noted that some bridge decks have overlays applied more than twice at stage 4 to maintain the acceptable deck condition in 100-year period. The results clearly show that the . by applying concrete sealers on bridge deck, and this effect is more significant for deck replacement and the 2nd deck overlay.

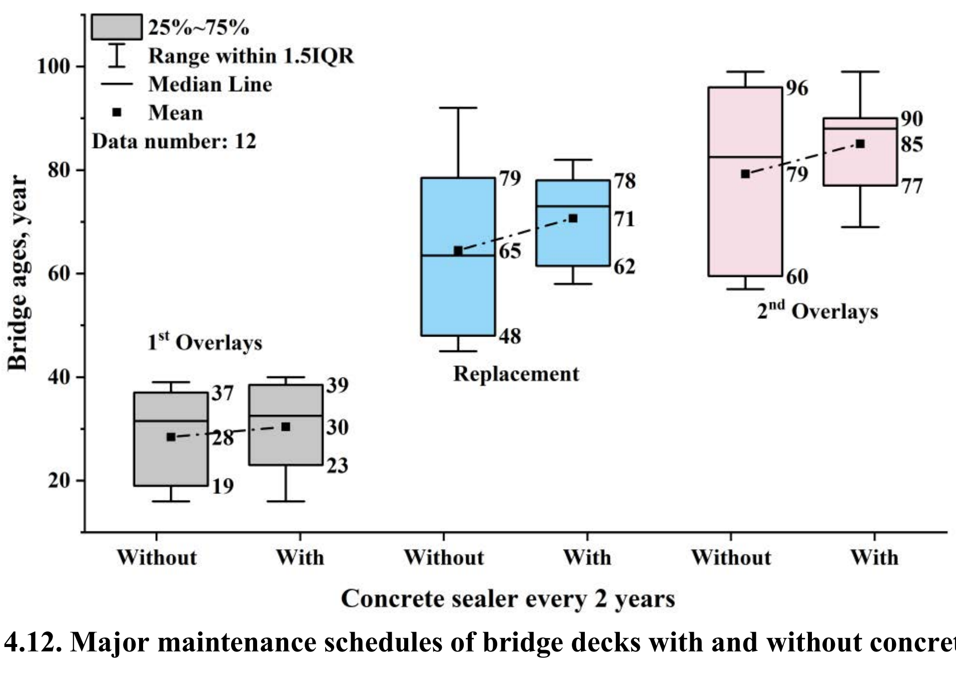


Figure 4.12. Major maintenance schedules of bridge decks with and without concrete sealers

Life cycle costs of bridge decks are calculated for service life of 100 years with monetary discount rate of 2%. Figure 4.13 presents the LCCA results for comparing the life cycle costs of 12 bridge decks with and without concrete sealer in 100-year lifespan. Based on the bridge database, it is possible that concrete sealers are applied every 2 or 4 years. One previous study in Oklahoma (Moradllo, Sudbrink, and Ley, 2016) found that concrete sealer was still effective after 12 years of application. Thus, the interval of concrete sealer is selected as 2, 4, and 12 years in the analysis.

The results show that the life cycle cost of bridge decks without concrete sealers is initially lower than those with concrete sealers due to the additional costs of concrete sealers. Since deck overlays and replacement are applied earlier for the bridge decks without concrete sealer, the life cycle costs increase quickly during the period of 45th to 50th years. The concrete sealer causes similar life-cycle costs when it is applied every 2 years. As the application intervals of concrete sealers increase, the life-cycle cost is further reduced. The accumulated life cycle cost after 100 years of service life decreased from around \$224/SF to \$211/SF (i.e., 6% reduction) on average when concrete sealers are applied every 12 years.

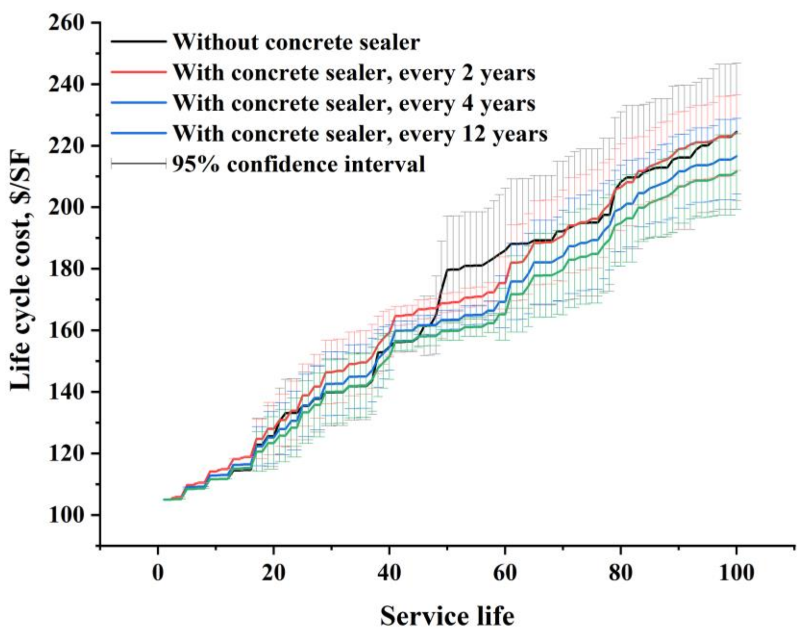


Figure 4.13. Life cycle costs for bridge decks with and without concrete sealers.

The monetary discount rate is important in LCCA to discount future costs for long-term projects. The equivalent uniform annual costs (EUACs) are calculated for all LCCA cases using different discount rates based on the range in 2023 Discount Rates for OMB Circular No. A-94. Figure 4.14 compares EUACs of bridge decks with and without concrete sealers at different scenarios. The results show that EUAC increases with the increase of discount rate. When the concrete sealer is applied every 2 years, the EUACs at all discount rates are close to those without concrete sealers. When concrete sealers are applied every 4 or 12 years, lower EUACs are observed and the difference of EUACs between the bridge decks with and without concrete sealer is enlarged as the discount rate increases.

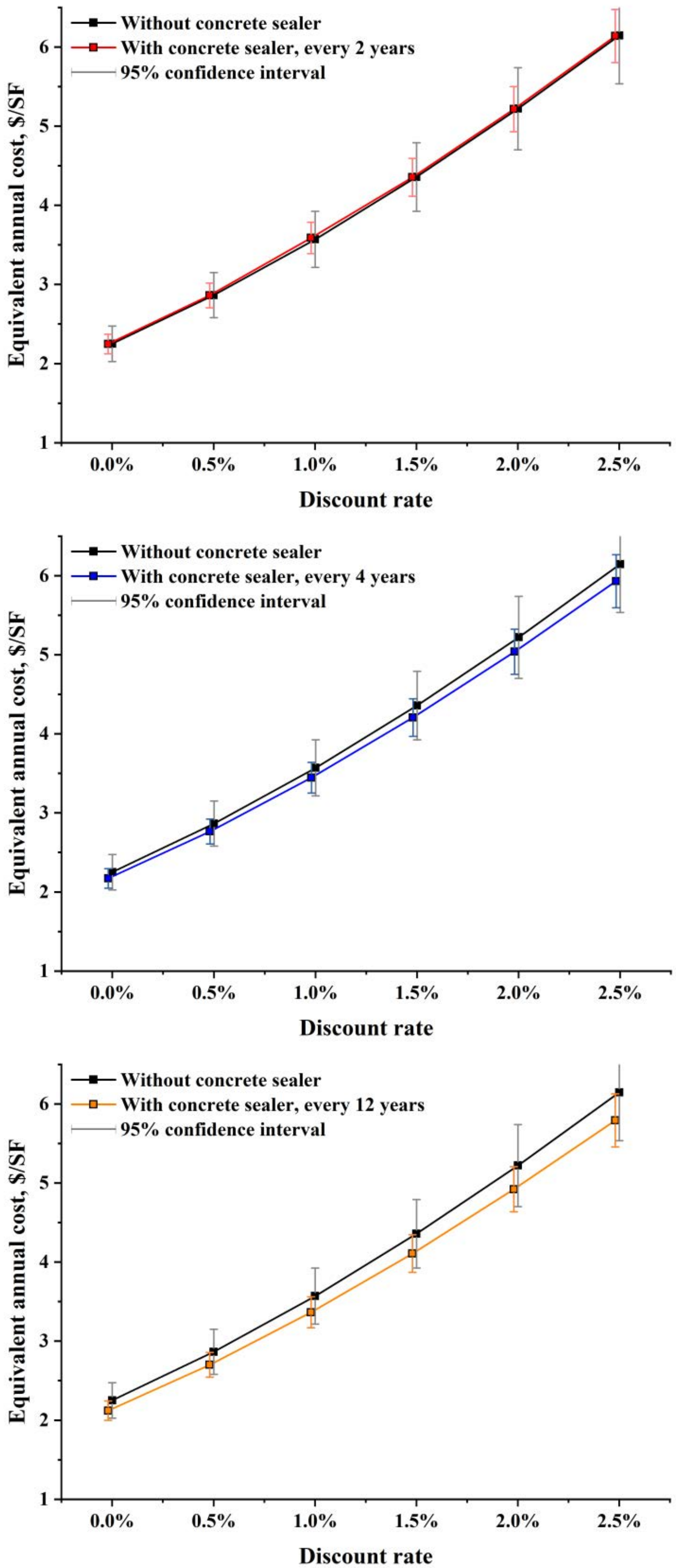


Figure 4.14. Equivalent uniform annual costs with concrete sealer at every (a) 2, (b) 4, and (c) 12 years as compared to the cases without concrete sealer

4.5 Summary

A total of 26,372 sets of bridge data with construction years from 1880 to 2021 were analyzed to investigate the impact of salt usage and surface coating (sealer and epoxy) on bridge performance. The deck maintenance history shows that the average bridge age at deck overlay and deck replacement is around 25 and 50 years, respectively. Deck overlay can improve deck condition from average rating of 6.3 to 7.1, and deck replacement can efficiently recover deck condition from average rating of 5.3 to 8.5. Regarding the winter maintenance data, the annual average salt usages (prewet salt, dry salt, and salt brine) vary significantly across different locations.

Random Forest machine learning models were developed to predict bridge deck ratings. The input parameters included bridge/deck age, application time of different preservation and maintenance activities, average daily traffic (ADT) data, annual average salt usage, and general bridge attributes (such as length, deck lane, deck materials). All four RF machine learning models show good accuracy for predicting bridge deck ratings with R-square values of 0.72-0.93. Bridge age and deck age presented the highest importance among all factors, followed by average daily traffic of car (ADT car). The importance of three types of salt usages is similar.

Two prediction scenarios were then created using the Random Forest models. The effect of concrete sealer on bridge deck condition is observable at stage 1, but its difference diminishes at later stages. This agrees with previous WHRP study 06-06 which noticed the importance of sealer reapplication. "Without reapplication, the sealers were ineffective over the long term." In terms of the type of salt usage, more usages of prewet salt and salt brine result in slightly higher deck condition ratings, while more dry salt usage presents slightly lower deck condition ratings, indicating the benefits of salt brine over dry salt.

Orthogonal analysis was conducted to further quantify the influence of concrete sealer and salt usage. When concrete sealer is applied at every 2 or 4 years, results indicate that applying concrete sealers can help extend service life of bridge deck by around 1~2 years at stage 1-3 and reduce the number of overlays at stage 4. As for salt usage, results indicate that more usage of salt brine can extend service life by around 1~2 years and reduce one overlay at stage 4.

Finally, life cycle cost analysis (LCCA) was performed using average cost from Bureau of Structures Cost Estimate Calculations of WisDOT. When concrete sealer is applied every 2 years, maintenance schedule can be delayed. However, the cost of concrete sealer cannot be neglected. The life cycle cost is similar when sealer is applied every 2 years. If concrete sealer is applied every 12 years, 6% reduction in life cycle cost could be achieved.

Chapter 5 Conclusions and Recommendations

5.1 Summary and Conclusions

The objectives of this project were to: (1) quantify the impact of applied anti-icing solutions on dry concrete surfaces, and (2) recommend countermeasures that would reduce the adverse impacts on concrete pavement and bridge deck durability.

Based on laboratory tests, accelerated field study at MnROAD, and analysis of performance data of Wisconsin pavements and bridges, the following conclusions were reached:

1. The concrete specimens had good freeze-thaw performance indicating samples were collected from a high-quality and durable field mixture (28-day compressive strength *f'c* 6500 psi for the A-FA concrete). This was also observed on concrete panels placed at MnROAD.
2. The anti-iced concrete samples had roughly half the amount of material scaled from the surface as the deiced concrete samples.
3. Chloride contents in the anti-iced concrete samples were also about half of the deiced concrete samples.
4. Silane surface treatment provided a significant reduction in chloride penetration of around 50% for most conditions and epoxy effectively blocking chlorides with low to no chlorides penetrated into the concrete.
5. When a 75-psi tire pressure was used to simulate field conditions and tested in the laboratory, chloride contents were up to 66% higher than no tire pressure. Chloride ingress may differ for higher or lower tire pressures.
6. Although no visual distress was observed on concrete panels at MnROAD due to the short study period (17 months), laboratory tests on core samples found lower chloride content on type A-FA concrete than on type A concrete, proving the benefits of fly ash. The silane treatment produced approximately a 50% reduction in chloride ingress with epoxy providing the greatest reduction.
7. In terms of the effect from traffic loads, the cored concrete samples from MnROAD showed that in all cases the chloride content in the wheelpath was greater (9% at surface and 41% at the lower depth) than the non-wheelpath samples, confirming with the hypothesis that tire pressure greatly increases the ingress of chloride to concrete.
8. The analysis of pavement management system data and salt data did not indicate any definitive relationship between the observed distress density and salt application rate. This may due to the complexity of field performance (confound results from many factors).
9. Random Forest machine learning models were developed to predict bridge deck ratings. The effect of concrete sealer on bridge deck condition is observable at stage 1, but its difference diminishes at later stages. Hence, the reapplication of concrete sealer is necessary to maintain its long-term effectiveness.
10. When concrete sealer is applied at every 2 or 4 years, the analysis results indicated that applying concrete sealers can help extend service life of bridge deck by around 1~2 years at stage 1-3 and reduce the number of overlays at stage 4.
11. As for salt usage, the analysis results indicated that more usage of salt brine for anti-icing (vs. solid salt for deicing) can extend service life by around 1~2 years and reduce one overlay at stage 4.
12. Life cycle cost analysis showed that maintenance schedule can be delayed when concrete sealer is applied every 2 years. However, the cost of concrete sealer cannot be neglected.

5.2 Recommendations

Based on analyses from laboratory tests, field study, and historical performance data of Wisconsin pavements and bridges, the following recommendations are proposed:

1. For the concrete mixtures tested and for the NaCl conditions employed for this study, anti-icing should be the preferred winter maintenance practice. Wisconsin has increased the usage of liquid brine in the past decade achieving faster time to bare/wet, higher friction, and less salt usage (Claros, et al., 2021). WisDOT can assist with more counties and municipalities to convert from rock salt to salt brine. This includes mixing equipment, storage facilities, tank trucks, and staff training.
2. The current policy about deicing and anti-icing application rate as stated in the *Highway Maintenance Manual - Chapter 6, Section 20 – Snow Removal Materials* is outdated (2012). WisDOT can update the application rate based on recent studies such as Clear Roads Project 19-01 (Claros, et al., 2021).
3. Penetrating sealer is a cost-effective treatment to extend the service life of bridge decks. We recommend WisDOT to continue implementing protective surface treatment to bridge decks according to WisDOT Standard Specification 502.3.13.2 *Protective Surface Treatment* and Construction and Materials Manual 525.3.3 *Protective Surface Treatment*.
4. Protective surface treatments have to be reapplied at a certain frequency to keep their effectiveness. The current (2021) WisDOT Bridge Manual, Chapter 42 – *Bridge Preservation* specifies sealing eligible concrete decks with sealant every 3~5 years. But according to WHRP 18-03 (Bektaş, et al., 2020), "eligible concrete decks on bridges that carry an annual average daily traffic (AADT) of 15,000 or higher should be sealed every three years." WisDOT needs to further investigate whether to adopt this recommendation or to keep the current frequency.
5. The concrete samples tested in this project are high quality, proving the effectiveness of optimized gradation and supplementary cementitious materials (SCM). Therefore, it is recommended to continue implementing concrete mix design with optimized gradation. Regarding the shortage of fly ash, WisDOT needs to find effective methods or replacements to maintain its benefits.

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