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Construction & Materials Best Practice for Concrete Sidewalks

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16. Abstract

This report summarizes an 18-month effort to investigate best practices to incorporate into the materials and construction of concrete sidewalks to mitigate surface scaling damage induced by freeze-thaw cycles in the presence of deicing chemicals. The study involved an in-situ experimental study accompanied by laboratory testing and quantitative analyses to determine key factors that impact sidewalk performance and durability. The primary variables considered in the study were concrete mixture design (aggregate/paste optimization, air content, and cementitious material replacements), workmanship (delivery, placement, finishing, curing), and deicing treatment.

The collective effort involved participants from construction companies, a concrete producer, academia, testing laboratories, and the Massachusetts Dept. of Transportation (MassDOT). Fifty-four unique sidewalk panels were placed adjacent to the Hopkinton, MA, MassDOT Research and Materials Laboratory. Collectively, the results indicate that sidewalk performance can be controlled though a combination of optimized mix design formulation, proper pre-placement, placement, finishing, curing, cold and hot weather concreting practices, contractor quality control, and department acceptance. Recommendations based on findings and in combination with referenced standards are provided, covering the range of variables studied in this research.

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Construction and Materials Best Practices for Concrete Sidewalks

Final Report

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This study was made possible through a combined partnership between the Massachusetts Department of Transportation, the University of Massachusetts, Amherst, and several members representing the concrete construction industry. The contributions of all the participants in this study are appreciated, particularly Construction Industries of Massachusetts (CIM), Massachusetts Concrete and Aggregate Producers Association (MaCAPA), ATC Group Services LLC, Barletta Company Inc., J.G. MacLellan Concrete Company Inc., McCourt Construction Company Inc., and SpecChem Cure Shield Ex.

Disclaimer

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Executive Summary

This study of Construction and Materials Best Practices for Concrete Sidewalks was undertaken as part of the Massachusetts Department of Transportation (MassDOT) Research Program. This program is funded with Federal Highway Administration (FHWA) State Planning and Research (SPR) funds. Through this program, applied research is conducted on topics of importance to the Commonwealth of Massachusetts transportation agencies.

This report summarizes an 18-month effort to investigate best practices to incorporate into the materials and construction of concrete sidewalks to mitigate surface scaling damage induced by freeze-thaw cycles in the presence of deicing chemicals. The study involved an in-situ experimental study accompanied by laboratory testing and quantitative analyses to determine key factors that impact sidewalk performance and durability. The collective effort involved participants from construction companies, a concrete producer, academia, testing laboratories, and MassDOT.

The primary variables considered in the study were concrete mixture design (aggregate/paste optimization, air content, and cementitious material replacements), workmanship (delivery, placement, finishing, curing), and deicing treatment. Mix designs featured a range of slag percentage (between 25% and 50%) and fly ash percentage (between 15% and 37%) to specifically investigate the role of cementitious material replacement in promoting or retarding scaling, resulting in six different mix designs. Three curing methods were investigated: no curing, a chemical curing/sealing agent, and saturated covers. Additionally, two deicing chemicals were employed: salt (NaCl) and a magnesium chloride ($MgCl_2$) solution. Fifty-four unique sidewalk panels were placed adjacent to the Hopkinton, Massachusetts, MassDOT Research and Materials Laboratory. A set of nine of these panels were selected for poor workmanship practices, to demonstrate the impact of these practices on scaling durability performance. Concurrent with placement, concrete cylinders and rectangular prism specimens were also cast for laboratory material testing. The experimental program was extensive, including testing of aggregate system (coarse and fine), fresh concrete properties, hardened concrete properties, scaling via *ASTM C672 (WITHDRAWN 2021)*, petrographic analysis and chloride content of extracted concrete cores, and development of a photogrammetry-based scaling analysis tool to assess the performance of the in-situ sidewalks over time.

Collectively, the results indicate that sidewalk performance can be controlled though a combination of optimized mix design formulation, proper pre-placement, placement, finishing, curing, cold and hot weather concreting practices, contractor quality control, and department acceptance. Recommendations based on findings and in combination with referenced standards are provided, covering the range of variables studied in this research.

Surface scaling of concrete sidewalks is largely influenced by the properties and strength of the top layer of concrete that extends a few millimeters into the body of the concrete sidewalk (typically 3 to 6 mm [0.12 to 0.2 inch]). This layer needs to have adequate strength, excellent air void structure, and low w/cm content to withstand the rigors of freeze-thaw cycles combined with application of deicing materials. To achieve durable performance of sidewalks against surface scaling, materials, construction procedures, and maintenance operations must all be closely quality controlled. Judicious mix design, quality control during production of concrete, finishing, curing, and protection of concrete through adequate curing are paramount for good performance. If proper care during the first winter is not adhered to, even high-quality concrete may be susceptible to scaling. The large number of organizations involved in this study reflects the collaborations that must be present in production of durable sidewalks in cold-weather regions.

To ensure a high quality and durable concrete sidewalk is produced and constructed, both contractor quality control (QC) and department acceptance must be conducted. QC must be established, maintained, and performed by the contractor (and sub-contractors) to monitor, assess, and adjust production and construction processes, maintain continuous control of the process, and ensure that the final material or product will meet the specified level of quality. QC must be incorporated into all stages of cement concrete sidewalk production and construction, including control, handling, and storage of constituent materials, mix design formulation, batching, mixing, transporting, sub-grade preparation, placement, finishing, curing, cold weather concreting, and hot weather concreting. Acceptance must be performed by the department to evaluate the degree of compliance with contract requirements, monitor contractor and sub-contractor QC activities, and determine the acceptability of all material produced and placed.

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List of Acronyms

Acronym	**Expansion**
AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
ASTM	American Society for Testing and Materials
CIM	Construction Industries of Massachusetts
FHWA	Federal Highway Administration
MaCAPA	Massachusetts Concrete and Aggregate Producers Association
MassDOT	Massachusetts Department of Transportation
NRMCA	National Ready Mixed Concrete Association
PCA	Portland Cement Association
SCM	Supplementary Cementitious Materials
UMass	University of Massachusetts Amherst
VOC	Volatile Organic Compound
w/cm	Water-Cementitious Material Ratio

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1.0 Introduction

The Construction and Materials Best Practices for Concrete Sidewalks research study was undertaken as part of the Massachusetts Department of Transportation (MassDOT) Research Program, through the combined partnership of the Construction Industries of Massachusetts (CIM), Massachusetts Concrete and Aggregate Producers Association (MaCAPA), the Massachusetts Department of Transportation (MassDOT), and the University of Massachusetts Amherst (UMass). This program is funded with Federal Highway Administration (FHWA) State Planning and Research (SPR) funds. Through this program, applied research is conducted on topics of importance to the Commonwealth of Massachusetts transportation agencies.

1.1 Problem Statement

MassDOT has observed numerous instances of significant early concrete sidewalk degradation and deterioration due to scaling throughout the Commonwealth, which has resulted in costly repairs or replacements. Contracts exhibiting concrete sidewalk scaling are identified in Table 1.1.

Table 1.1: Contracts exhibiting concrete sidewalk scaling

Contract	District	Location
77717	4	Belmont-Watertown
78958	4	Danvers
84360	2	Barre
84978	2	Belchertown
85012	4	Beverly
90474	3	Medway
90574	2	Springfield
90728	4	Salem
97278	2	Longmeadow
97686	2	Warren
97770	4	Lexington
100596	6	Boston
101888	4	Lowell
101985	3	Fitchburg-Leominster-Lunenburg
102133	4	Revere

Figure 1.1: Examples of scaling degradation on Contract 90474, Medway (left), and Contract 101985, Fitchburg-Leominster-Lunenburg (right)

1.2 Objective

This research study is intended to identify and recommend best practices to incorporate into the materials and construction of concrete sidewalks to promote long-term durability and to prevent premature deterioration.

1.3 Mechanisms of Scaling

The Portland Cement Association (PCA) defines scaling as a general loss of surface mortar or mortar surrounding the coarse aggregate particles on a concrete surface. The mechanism behind concrete scaling involves the phenomenon of water in the concrete pore solution solidifying into ice. As the temperature drops below freezing, the pore water freezes and increases by 9% in total volume, causing internal pressures inside the concrete. Dissolved chlorides in the pore solution are also expelled from the solution as it freezes, causing even more internal pressures. Concrete surfaces with a weak top layer of cement, known as laitance, are susceptible to scaling due to the internal pressures exceeding the strength of the laitance. Figure 1.2 illustrates the effect of bleed water in the creation of a weak top layer that may lead to surface scaling.

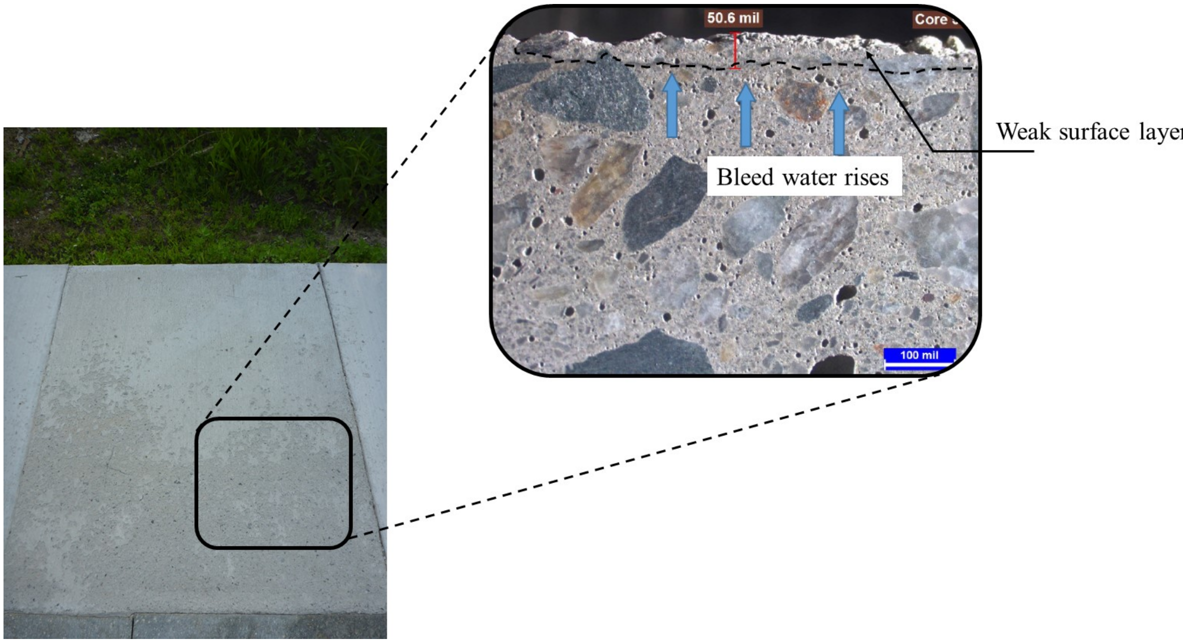

Figure 1.2: Mechanism of surface scaling

Laitance is attributed to concrete that lacks any of the following activities or properties:

- Workmanship
- Workability
- Maturity
- Durability

1.3.1 Workmanship

Workmanship is defined as the degree of skill with which a material, product, or structure is made. The finished material, product, or structure is only as good as the workmanship that goes into it. The quality of the workmanship is also affected by the quality of the material being delivered to the job site. Therefore, it is vital for the contractor and the cement concrete producer to maintain an open line of communication, so that issues in workmanship are addressed and resolved immediately.

In addition to communication, training, qualification, and certification of craftsmen are also important components of workmanship, to provide craftsmen with the best practices required to construct concrete sidewalk to prevent premature deterioration. Concrete sidewalks constructed with poor workmanship are more susceptible to scaling due to:

- Lack of personnel training for concrete flatwork
- Insufficiently sized work crew
- Use of prohibited tools
- Improper strike off techniques
- Improper practice of adding additional water to the concrete surface

- Finishing bleed water back into the concrete surface
- Excessive finishing and working of the concrete surface
- Improper or lack of identification of time of set
- Lack of proper curing techniques
- Lack of protection of early-age concrete from deleterious environmental conditions

Improper workmanship practices negatively affect the performance and durability of the cement concrete, which will inevitably lead to scaling.

1.3.2 Workability

Workability is defined as the "property of freshly mixed concrete or mortar that determines the ease with which it can be mixed, placed, consolidated, and finished to a homogenous condition" *(1).* Several characteristics of fresh concrete are related to workability, including consistency, fluidity, response to vibration, mobility, pumpability, compactibility, finishability, and harshness *(2).* Workability is affected by many factors, including water content, aggregate, air-entrainment, time, temperature, cement, supplementary cementitious materials, and chemical admixtures. Concrete designed, produced, or constructed with poor workability is more susceptible to scaling due to:

- Lack of optimized combined gradations and admixtures that promote workability incorporated into the mix design, resulting in the improper practice of adding additional water to compensate for lack of workability
- Lack of protection from environmental conditions, such as hot ambient temperatures, resulting in excessive finishing and working of the concrete surface

1.3.3 Maturity

The maturity of the concrete is directly correlated to the concrete's ability to withstand the rigors of freezing, thawing, and deicing cycles. Immature concrete is more susceptible to scaling due to its high water content, hydraulic pressures, incomplete hydration process, and lack of compressive strength. Proper workmanship practices, including curing and protection, are required to ensure that the concrete has sufficiently matured to prevent premature deterioration due to scaling.

1.3.4 Durability

Durability is defined as "the concrete's ability to resist chemical and environmental attack, including scaling, throughout the entire duration of its life cycle" *(1)*. The concrete's ability to be durable and resist freezing, thawing, and deicing damage is dependent on the concrete's permeability, air void system, finishing operations, and curing operations. As the concrete's permeability decreases, the concrete's resistance to scaling increases. Decreased concrete permeability can be achieved through a decreased water-cementitious ratio (w/cm) and an increased rate of hydration with the incorporation of supplementary cementitious materials (SCM) and proper curing. The concrete's air void system should contain adequately entrained, sized, and distributed air bubbles to provide space for freezing, thawing, and deicing cycles. Concrete designed, produced, or constructed with poor durability is more susceptible to scaling, due to poor air entrainment, high water-cementitious ratio, low hydration, or weak surface layer (laitance) due to excessive bleeding from increased w/cm ratio, poor finishing, or lack of curing.

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2.0 Scope

The following variables were applied to this research project, to investigate their potential effects on concrete sidewalk durability as it relates to scaling, through sampling, testing, inspection, and computer-based photogrammetry.

- Mix Design Formulations
- Workmanship Practices
- Curing Methods
- Deicing Agent Applications

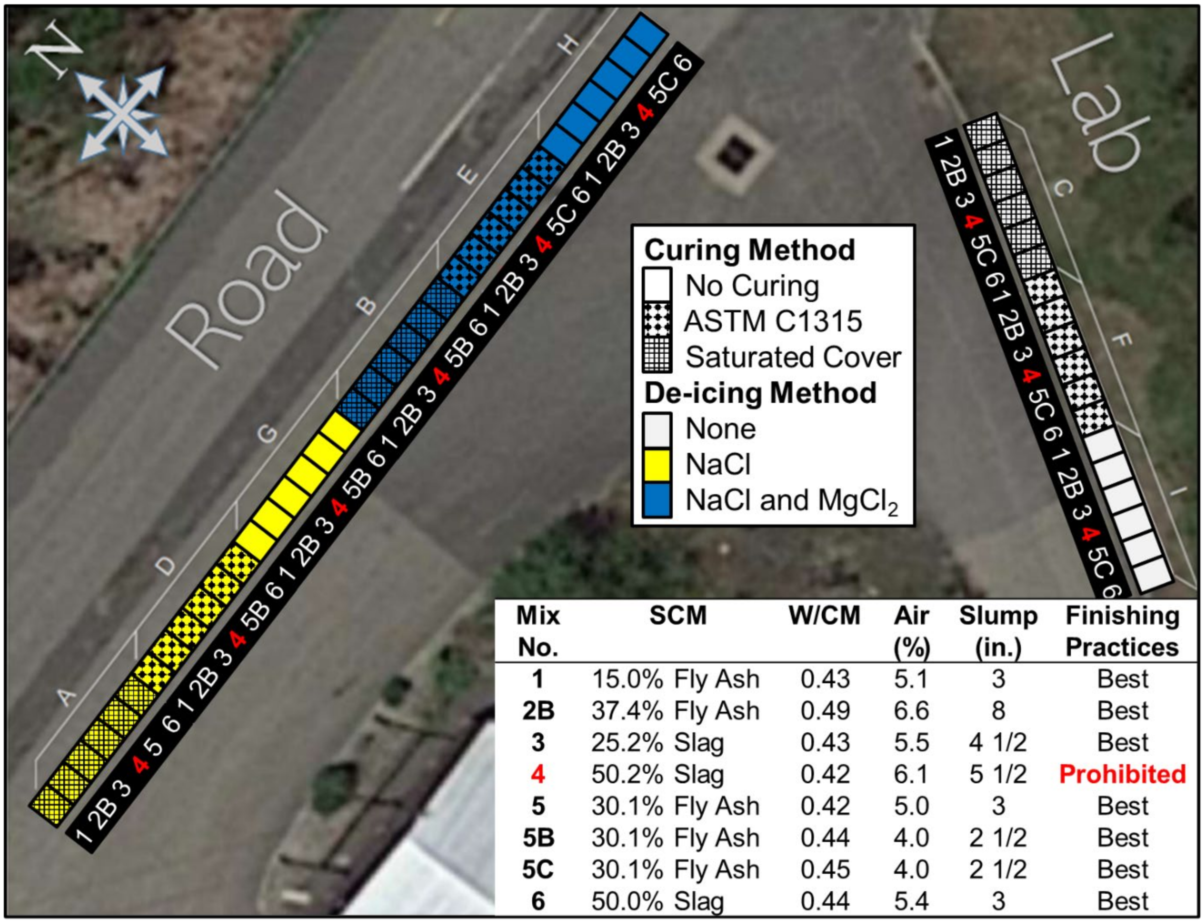
The effects of the variables were studied and evaluated on 54 cement concrete sidewalk panels (4 x 5 ft., 6 in. nominal thickness) that were constructed specifically for this research project at the Snow and Ice Maintenance facility in Hopkinton, Massachusetts (referred as the test site for this study). Panel Groups A, D, G, B, E, and H were constructed on grade, directly abutting the existing asphalt pavement, to be exposed to harsher deicing conditions than Panel Groups C, F, and I. Panel Groups C, F, and I were constructed behind existing granite curbing, above the level of the existing asphalt pavement, to simulate real-world conditions. The participants identified in Table 2.1 provided their materials and services to this research project.

Table 2.1: Participants

Entity	Participant
MassDOT	Construction Section; District 2; District 3; Research and Materials Section
Academia	University of Massachusetts (UMass) Amherst
Petrographic Examination Laboratory	Wiss, Janney, Elstner Associates, Inc.
Industry Field Testing Laboratory	ATC Group Services, LLC
Industry Associations	Construction Industries of Massachusetts (CIM); Massachusetts Concrete and Aggregate Producers Associations (MaCAPA)
Excavator Company	Barletta Co., Inc.
Cement Concrete Producer	J. G. MacLellan Concrete Co., Inc.
Construction Company	McCourt Construction Co., Inc.
Curing and Sealing Compound Product	SpecChem Cure Shield Ex

The cement concrete sidewalk exposure site plan locating the variables applied to each panel is provided in Figure 2.1 (refer to applicable sections of this report for more details regarding the variables). The different participants of the study contributed materials to this report.

Information provided by participants other than UMass Amherst is indicated in each relevant section throughout the research report.



Mix No.	SCM	W/CM	Air (%)	Slump (in.)	Finishing Practices
1	15.0% Fly Ash	0.43	5.1	3	Best
2B	37.4% Fly Ash	0.49	6.6	8	Best
3	25.2% Slag	0.43	5.5	4 1/2	Best
4	50.2% Slag	0.42	6.1	5 1/2	Prohibited
5	30.1% Fly Ash	0.42	5.0	3	Best
5B	30.1% Fly Ash	0.44	4.0	2 1/2	Best
5C	30.1% Fly Ash	0.45	4.0	2 1/2	Best
6	50.0% Slag	0.44	5.4	3	Best

Figure 2.1: Cement concrete sidewalk exposure site plan

2.1 Mix Design Formulations

Six unique mix designs were proposed by the cement concrete producer listed in Table 2.1 with varying quantities of fly ash and slag, to identify any effects that the supplementary cementitious materials (SCMs) may have on the cement concrete sidewalk's performance. Mix design formulations are identified in Chapter 3.0. The cement concrete exposure site plan (Figure 2.1) identifies the mix design formulation produced for each panel cast. UMass Amherst did not participate in the concrete mixture development, since one of the goals was to use materials that are widely used in industry for sidewalk fabrication. Mixes 2, 4, 5, and 6 had a higher volume of fly ash or slag than the volumes in mixes 1 and 3. The batching quantity of cement in Mix 2 was lower than the design value, resulting in a higher percentage of fly ash replacement of 37% instead of the target 30%. Because of this batching error, the as-delivered Mix 2 is referred to as Mix 2B in this report.

2.2 Pre-Placement, Placement, and Finishing Practices

The National Ready Mixed Concrete Association's (NRMCA) *Best Practices for Exterior Flatwork Finishing Manual* was used to identify "best" and "prohibited" pre-placement, placement, and finishing practices for this study. Best practices have been developed by NRMCA to mitigate the effect of surface scaling of concrete and were therefore considered the baseline for assessment in this study. The cement concrete sidewalk exposure site plan (Figure 2.1) identifies the proposed finishing practice that was to be applied to each panel cast. Numbered subsections in the following paragraphs refer to the *MassDOT Standard Specifications for Highways and Bridges* (*MassDOT Standard Specifications*) *(3).*

2.2.1 Best Pre-Placement Practices

All 54 cement concrete sidewalk panels were to be prepared and constructed following the NRMCA best practices for pre-placement. Requirements for best practices from NRMCA, also adopted in the *MassDOT Specifications (3),* are repeated here for completeness.

2.2.1.1 Pre-Placement Requirements

Excavation of the area shall be in accordance with the applicable portions of *MassDOT Specifications,* "Subsection 120: Excavation".

The subgrade for the sidewalks and driveways shall be shaped parallel to the proposed surface of the sidewalks and driveways and thoroughly compacted. All depressions in the subgrade shall be filled with suitable material and again compacted until the surface is smooth and hard. Prior to the placement of the subbase, the Contractor shall inspect the prepared subgrade to ensure that it is in conformance with the required grade and cross-section. Subgrade shall be fine graded to meet the applicable requirements of *MassDOT Specifications*, "Subsection 170: Grading."

After the subgrade has been prepared, a gravel subbase shall be placed upon it. After being compacted thoroughly, the subbase shall be at least 8 inches thick and parallel to the proposed surface of the sidewalk. Prior to the placement of the cement concrete, the Contractor shall inspect the prepared subbase material to ensure that it is in conformance with the required grade and cross-section. Subbase material that is not in accordance with the plans or specifications shall be reworked or replaced to meet the applicable requirements of *MassDOT Specifications,* "Subsection 170: Grading before the start of cement concrete placement." When placing cement concrete, the compacted subbase shall not be frozen or have standing water.

Side form and transverse form shall be smooth, free from warp, of sufficient strength to resist springing out of shape, of a depth to conform to the thickness of the proposed sidewalk or pedestrian curb ramp and of a type satisfactory to the Engineer.

All mortar or dirt from previously used forms shall be completely removed prior to use. The forms shall be well staked and thoroughly graded and set to the established lines with their upper edge conforming to the grade of the finished sidewalk or pedestrian curb ramp which shall have sufficient pitch to the roadside edge to provide for surface drainage.

All pedestrian curb ramp joints and transition sections which define grade changes shall be formed staked and checked for dimension, grade and slope conformance prior to placing cement concrete.

All forms shall be oiled before placing concrete.

Checklist Items

- Apply sufficient base compaction
- Moisten sub-base, free of standing water
- Secure forms, straight and level
- Mark expansion locations
- Avoid prohibited practices including, placement on frozen sub-grade

2.2.2 Best Placement Practices

All 54 cement concrete sidewalk panels were to be subjected to the NRMCA best practices for placement. Requirements for best practices from NRMCA, also adopted in the *MassDOT Specifications (3),* are repeated here for completeness.

2.2.2.1 *Placement Requirements*

Placement of the concrete sidewalk panels was to be in accordance with *Interim Subsection 701: Cement Concrete Sidewalks, Pedestrian Curb Ramps, and Driveways*, which was being developed at the initiation of this project for incorporation into the 2021 *MassDOT Standard Specifications (3).* The latest draft version of Interim Subsection 701 was written in late 2020. Best practices recommended by the National Ready Mixed Concrete Association (NRMCA) were also to be followed. The following information is excerpted from these documents as applicable to this project.

The concrete shall be placed in alternate slabs 30 ft long except as otherwise ordered. The slabs shall be separated by transverse preformed expansion joint filler ½ in. thick.

Preformed expansion joint filler shall be placed adjacent to or around existing structures as directed. On the foundation, the concrete shall be placed in such quantity that after being thoroughly consolidated in place it shall be 4 in. deep. At driveways, the sidewalks shall be 6 in. deep. In conveying the concrete from the place of mixing to the place of deposit, the operation shall be conducted in such a manner that no mortar will be lost, and the concrete shall be so handled that the concrete will be of uniform composition throughout, showing neither excess nor lack of mortar in any one place. The surface of all concrete sidewalks shall be uniformly scored into block units of areas not more than 36 ft^2. The depth of the scoring shall be at least ½ in. deep and no more than ½ in. wide.

Checklist Items

- Direct concrete trucks to placement locations
- Handle chute discharge and truck movement
- Assist in preparing concrete for testing
- Direct trucks to washout area
- Localize placement to minimize moving material
- Level concrete in front of the screed
- Operate come-alongs or flat headed shovel to move concrete in form
- Consolidate concrete along form edge to avoid honeycombing
- Operate screed over top of forms in sawing action for surface leveling
- Operate magnesium bull float to push coarse aggregate below the surface and fill in the low spots or depressions
- Avoid prohibited practices including, not meeting AASHTO M 157 requirements, more than one addition of water prior to placement, exceeding the mix design formulation's w/cm content with additional water, toothed raking, dragging of internal vibrator, and internal vibrator to move concrete, and steel troweling or floating

2.2.3 Best Finishing Practices

Forty-five of the 54 cement concrete sidewalk panels containing the following mix design formulations were to be subjected to best finishing practices:

- Mix No. 1; Mix No. 2; Mix No. 3; Mix No. 5; and Mix No. 6

2.2.3.1 *Finishing Requirements*

Finishing was to be conducted following best practice recommendations included in this section for the 45 panels fabricated using the mixes listed above. Best finishing practices are taken from *MassDOT Standard Specifications (3)* and NRMCA best practices.

The finishing of concrete surface shall be conducted by qualified craftsmen. No finishing operation shall be performed while free water is present. Finishing operations shall be delayed until all bleed water and water sheen has left the surface and the concrete has started to harden. After water sheen has disappeared, edging operations, where required, shall be completed. After edging and joining operations, the surface shall be floated. Magnesium floats shall be used for all finishing operations. If necessary tooled joints and edges shall be rerun before and after floating to maintain uniformity. After floating, the surface shall be brushed by drawing a soft-bristled push broom with a long handle over the surface of the concrete to produce a nonslip surface.

Checklist Items

- Permit bleed water to dissipate and concrete to set
- Operate a hose drag or squeegee to remove water from the surface
- Check surface for flatness, fill/cut as necessary
- Finish surface with magnesium float
- Apply pulled broom finish at proper time to acceptable texture
- Clean broom when excessive mortar adheres
- Remove excess water from broom before use
- Finish edges and joints
- Finish well formed, properly spaced joints to sufficient depth
- Avoid prohibited practices including, steel troweling or floating, adding water to the surface, excessive working of surface, and pushing broom across surface

2.2.4 Prohibited Finishing Practices

Nine of the 54 cement concrete sidewalk panels were to be subjected to prohibited finishing practices, to identify any effects that the varying practices may have on the cement concrete sidewalk's performance. Prohibited finishing practices include the use of steel trowels, adding water to the surface (blessing) during finishing, and incorporating bleed water back into the concrete. These panels were intentionally finished using prohibited practices to observe the detrimental effect these practices have on surface scaling durability.

2.3 Curing Methods

The cement concrete sidewalk panels were to be subjected to varying curing methods, to identify any effects that the varying curing methods may have on the cement concrete sidewalk's performance. The cement concrete exposure site plan (Figure 2.1) identifies the curing method that was to be applied to each panel cast. The curing methods were selected collaboratively between MassDOT personnel and industry associations listed in Table 2.1.

2.3.1 Saturated Covers

The contractor was instructed to apply saturated covers to the panels identified in Figure 2.1, as required herein. However, the curing procedures conducted by the contractor did not fully comply with the requirements specified in this section. The curing conditions that the panels were subjected to are identified in Section 5.2.

2.3.1.1 *Materials Requirements*

Saturated covers used for final curing methods shall meet AASHTO M 182, Class 3. Saturated covers shall be in good condition, free from holes, tears, or other defects that would render it unsuitable for curing cement concrete and cementitious materials. Saturated covers shall be dried to prevent mildew when storing. Prior to application, saturated covers shall be thoroughly rinsed in water and free of harmful substances that are deleterious or cause discoloration to cement concrete and cementitious materials. Saturated covers shall have sufficient thickness and proper positioning onto the surface to maximize moisture retention. Saturated covers shall contain adequate moisture to prevent moisture loss from the surface of cement concrete and cementitious materials. Saturated covers shall have the ability to retain sufficient moisture from continuous watering so that a film of water remains on the surface of cement concrete and cementitious materials throughout the entire duration of the final curing method cycle. Saturated covers shall not absorb water from cement concrete and cementitious materials. Polyethylene film may be applied over the saturated cover to limit the amount of continuous watering required for sufficient moisture retainage. Saturated covers shall accommodate uniform and slow drying of cement concrete and cementitious materials surfaces immediately prior to removal.

2.3.1.2 *Application Requirements*

Curing water shall be free of deleterious impurities, causing staining and deterioration. Curing water shall not exceed a temperature differential of more than 20°F from the internal concrete temperature, to prevent cracking due to temperature gradients causing strain that exceeds the strain capacity of concrete. Curing water shall remain above freezing temperatures throughout the duration of the curing cycle.

Final curing materials and procedures shall be applied to the concrete surface immediately after application of initial and intermediate curing materials, finishing operations, and final set of cement concrete, to prevent the loss of moisture and surface drying.

Materials used for final curing methods of cement concrete shall accommodate all exposed cement concrete surfaces with a continuous application of moisture throughout the entire duration of the final curing method cycle and provide controlled and gradual termination of the final curing method cycle.

Final curing materials applied to the concrete shall allow the concrete to mature sufficiently to achieve its designed and desired properties, including strength, volume stability, permeability, durability, and resistance to freezing, thawing, and deicing cycles. Insufficient application of final curing materials results in decreased strength and durability of the top surface of concrete.

Protection to the concrete surface and curing materials shall be required in instances where adverse weather conditions are present, until curing operations can be initiated without damaging the surface of the concrete.

Final curing materials and procedures shall be applied to the concrete surface throughout the entire duration of the curing cycle and meet minimum sustained temperature, duration, and strength requirements, as specified herein. Controlled and gradual termination of the final curing method cycle shall begin only after all specified conditions are met, until the concrete gradually cools to within 20°F of the ambient temperature.

2.3.1.3 *Temperature Protection Requirements*

The Contractor shall meet minimum sustained temperature, duration, and strength requirements throughout the entire duration of the curing cycle, as specified herein. Controlled and gradual termination of the curing cycle shall begin after all specified conditions are met (see Table 2.2).

Table 2.2: Termination of curing cycle

Sustained Concrete Temperature	**Final Curing Cycle Duration**	**Compressive Strength**
50°F ≤ °F ≤ 90°F	≥ Seven (7) days	≥ 70% f'_c

Note: Compressive strength cylinders for termination of curing cycle shall be cast and field cured under the same environmental conditions that the sidewalk is subjected to throughout the entire duration of the final curing cycle.

The Contractor shall conduct cold weather concreting procedures, operations, materials, and equipment required for the mixing, delivery, placement, finishing, curing, and protection of concrete, while surfaces are exposed to air temperatures falling below, or expected to fall below 40°F in accordance with ACI recommendations and specifications. All procedures, operations, materials, and equipment required for adequate protection and curing shall be present and ready for use prior to concrete production.

2.3.2 Liquid Membrane-Forming Compounds for Curing and Sealing

The Contractor was instructed to apply a liquid membrane-forming compound for curing and sealing to the panels identified in Figure 2.1, as required herein. However, the curing procedures conducted by the contractor did not fully meet the requirements specified in this section. The curing conditions that the panels were subjected to are identified in Section 5.2.

2.3.2.1 *Materials Requirements*

Liquid membrane-forming compounds for curing and sealing shall meet *ASTM C1315* Type I (clear or translucent) Class A (non-yellowing) and the Manufacturer's instructions and recommendations, and the requirements specified herein. In addition to moisture-retention capabilities, compounds shall exhibit specific properties, including alkali resistance, acid resistance, adhesion-promoting quality, and resistance to degradation by ultraviolet light.

2.3.2.2 *Application Requirements*

In addition to the requirements specified herein, application of liquid membrane-forming compounds for curing and sealing shall also meet the applicable requirements of Section 2.3.1.2.

Compounds shall form a continuous, non-yellowing, and durable film with quality moisture-retention properties. Compounds shall maintain the relative humidity of the concrete surface above 80% for seven days to sustain cement hydration. Compounds shall not affect the original color of the concrete surface. Compounds shall not degrade due to exposure to ultraviolet light from direct sunlight. Compounds shall meet the local and federal allowable Volatile Organic Compound (VOC) content limits.

White-pigmented compounds shall be used in instances where solar-heat gain is concern to the concrete surface. White-pigmented compounds shall be agitated in the container prior to application to prevent pigment from settling out resulting in non-uniform overage and ineffective curing.

Careful considerations shall be made by the Contractor to determine if the evaporation rate is exceeding the rate of bleeding, thus causing the surface to appear dry even though bleeding is still occurring. To diagnose and prevent this condition, the Contractor may place a transparent plastic sheet over a test area of the uncured and unfinished concrete surface and shall determine if any bleed water accumulates under the plastic. Under such conditions, the application of liquid membrane-forming compounds to the concrete surface shall be delayed to prevent bleed water from being sealed below the concrete surface, map cracking of the membrane films, reduction in moisture-retention capability, and the need for reapplication of the compound.

Prior to use, compounds shall be thoroughly mixed, stirred, and agitated per the Manufacturer's instructions and recommendations. Compounds shall be applied continuously and uniformly to the surface of the concrete per the Manufacturer's instructions and recommendations. Compounds shall be applied immediately after the disappearance of the surface water sheen following final finishing. Applying the compound immediately after final finishing and before all free water on the surface has evaporated will help prevent the formation of cracks. When using compounds to reduce moisture loss from formed surfaces, the exposed surface shall be wetted immediately after form removal and kept moist until the curing compound is applied. The concrete shall be allowed to reach a uniformly damp appearance with no free water on the surface, and then application of the compound shall begin at once. Delayed application will result in surface drying, absorption of the compound into the concrete, and no forming of a continuous membrane.

The concrete surface shall be damp when the compound is applied. Power-driven spray equipment shall be used for uniform application of compounds on large paving projects. Spray nozzles recommended by the compound Manufacturer and use of windshields shall be arranged by the Contractor to prevent wind-blown loss of compound and to ensure proper coverage application rates are achieved. The compound shall be applied by power sprayer, using appropriate wands and nozzles with pressures between 25 and 100 psi. The Contractor shall fill the power sprayer with curing compound from the Manufacturer's original container in the presence of the Engineer. Any dilution as recommended by the Manufacturer shall take place in the presence of the Engineer. For very small areas such as repairs, the compound shall be applied with a wide, soft-bristled brush or paint roller.

The Contractor shall verify the application rate and procedures are in accordance with the Manufacturer's instructions and recommendations. At least one uniform coat shall be applied at a rate of 150 to 200 ft^2/gallon. On very deeply textured surfaces, the surface area to be treated shall be at least twice the surface area of the surface. In such cases, two separate applications may be needed, each at 200 ft^2/gallon or greater if specified by the Manufacturer to achieve the desired moisture retention rate, with the first being allowed to become tacky before the second is applied. If two coats are necessary to ensure complete coverage, for effective protection the second coat should be applied at right angles to the first. Complete coverage of the surface shall be attained due to the potential for formation of small pinholes in the membrane, which will result in loss of moisture from the concrete. Compounds shall not sag, run off peaks, or collect in grooves.

Compounds and procedures shall be compatible with concrete surfaces receiving subsequent applications or placements of concrete, overlays, coatings, paints, sealers, finishes or other toppings to ensure acceptable bonding to the concrete. Testing to establish compatibility among the curing compound, subsequent surface treatments, concrete moisture content and the actual finished surface texture of the concrete shall be conducted when compatibility is not known. The compound Manufacturer shall be consulted by the Contractor to determine the compatibility of the application. Compounds shall not be applied to concrete surfaces where bonding of subsequent applications or placements is incompatible or is of concern. The use of wax-based curing compounds shall be prohibited in instances where concrete surfaces are subject to additional toppings and vehicular, pedestrian, or other traffic.
Deliberate removal of compounds in the presence of the Engineer and in accordance with Manufacturer's instructions and recommendations shall be conducted as an alternative to compatibility testing, incompatibility, or in instances where bonding is of concern. Bonding of subsequent materials may still be inhibited by the presence of the compound even after the moisture retention characteristics of the compound have diminished.

2.3.2.3 *Temperature Protection Requirements*

The Contractor shall meet the temperature protection requirements specified in Section 2.3.1.3.

2.3.3 No Curing

The Contractor was instructed to refrain from applying curing methods to the panels identified in Figure 2.1. These panels were to be subjected only to the ambient weather conditions surrounding the panels throughout the duration of the curing cycle and beyond. The ambient weather conditions that the panels were subjected to are identified in Section 5.0.

2.4 Deicing Agent Applications

The cement concrete sidewalk panels were subjected to varying deicing agent applications, to identify any effects that the varying deicing agent applications may have on the cement concrete sidewalk's performance. The cement concrete exposure site plan (Figure 2.1) identifies the deicing agent applied to each panel at the test site.

2.4.1 Sodium Chloride Application

MassDOT deicing trucks applied sodium chloride (NaCl) to the panels identified in Figure 2.1 during snow and ice events. The NaCl application was not controlled intentionally; it followed the deicing protocols that MassDOT uses to treat roads during ice and snow events. The research study was purposely planned to mimic realistic in-service conditions to assess their impact on concrete sidewalk durability.

2.4.2 Magnesium Chloride Application

UMass Amherst periodically applied magnesium chloride ($MgCl_2$) to the panels identified in Figure 2.1, according to the manufacturer's recommendations. Information on the $MgCl_2$ product use for this study is provided in Appendix C, and a guide for its application is in Appendix D. Magnesium chloride was applied to the panels identified in Figure 2.1 on the following dates:

November 30, 2019
December 18, 2019
January 7, 2020
January 17, 2020
January 24, 2020
February 4, 2020
February 21, 2020
March 4, 2020

Sidewalk Groups B, E, and H were given periodic applications of magnesium chloride. Starting November 30, 2019, a solution of magnesium chloride ($MgCl_2$) was applied approximately every two weeks. When a snow or ice event was forecast within five days of the scheduled application, $MgCl_2$ was applied early in order to have a fresh coat on the panels before the ice or snow event.

The last application of $MgCl_2$ took place on March 4, 2020. Shortly afterward, UMass Amherst asked researchers to suspend nonessential research activities because of the coronavirus pandemic. On March 16, 2020, and March 24, 2020, the Governor of Massachusetts imposed a statewide stay-at-home order. This order stayed in place through May 1, 2020, when winter treatment was no longer necessary. The last date of $MgCl_2$ application was close enough to the originally planned end date (end of March), so it was not considered detrimental to the results of the study.

The $MgCl_2$ solution used was the 30% concentrate ProMelt MAG Inhibitor from Innovative Surface Solutions (see Appendix C). The Minnesota guidelines suggested by a representative from Innovative Surface Solutions recommends 0.2–0.4 gallons of $MgCl_2$ be applied per 1,000 ft^2 of pavement. The solution is typically applied using a truck or small construction vehicle. Because of the small area of the panels on the site, the research team used a hand pump sprayer, the RL Flo-Master Bleach Sprayer, pictured in Figure 2.2. At full pressure and 50° F, the pump applies about 5 oz. of $MgCl_2$ per minute. Thus, a concentration of 0.3 gallons of $MgCl_2$ per 1,000 ft^2 was applied on each panel by constant spraying for about 15 to 20 seconds in even coats. This rate decreased slightly when the $MgCl_2$ was colder (higher viscosity); the application time was therefore adjusted on colder days to ensure a similar concentration during application. Figure 2.3 shows Panel B2 after application of ProMelt MAG Inhibitor.

Figure 2.2: Flo-Master hand pump chemical applicator for $MgCl_2$

Figure 2.3: Panel B2 after application of $MgCl_2$ (December 18, 2019)

2.4.3 Residual Sodium Chloride Exposure

MassDOT deicing trucks applied NaCl to the entire asphalt pavement area of the Hopkinton Snow and Ice Maintenance Facility during snow and ice events. Due to vehicular traffic and snow removal, the residual NaCl from the asphalt pavement was applied to the panels identified in Figure 2.1.

Despite being constructed against granite curbing and not being directly targeted with deicing agents, the panels in Groups C, F, and I were still subjected to chloride-laden ice and snow pushed from roadway snow removal operations at the site. In fact, when visiting the site after a snow event, the researchers noted that snow and slush from the road was often plowed up onto these sidewalks and would then remain on the sidewalk surface until either the snow melted or the snow was shoveled in preparation for photographic documentation of the sidewalk panels (Figure 2.4). Parts of these eastern panels were in contact with salt for a longer duration than panels on the north side of the site, as snowmelt and salt more easily ran off the panels due to street grading.

(a) (b)

(c) (d)

Figure 2.4: Residual ice and snow remaining on panels in Groups C, F, and I after road snow removal Jan. 2 and 24, 2020]: (a) Panels C1 and C2 containing ice; (b) overall view of sidewalks after snow/ice removal; (c) and (d) panels C4 and C6 containing leftover salts after ice melted

3.0 Mix Design Formulation

The cement concrete mix design formulations incorporated into this study were batched and produced by the cement concrete producer (Table 2.1), with the sources and quantities of constituent materials identified in Tables 3.1 through 3.4. Mix design formulations are analyzed in this chapter, to determine the potential properties of the cement concrete. The mix design sheets are provided in Appendix G. This chapter provides mix quantities as delivered to the site or modified at the site through the addition of water.

Table 3.1: Aggregate

Material	Manufacturer	Location	Description	AASHTO
Fine	Ossipee Aggregates	Ossipee, NH	Normal Weight	M 6
3/8 in.	Aggregate Industries	Littleton, MA	Normal Weight - 8	M 80
3/4 in.	Aggregate Industries	Littleton, MA	Normal Weight - 67	M 80

Table 3.2: Hydraulic cement and supplementary cementitious materials

Material	Manufacturer	Location	Type	Description	AASHTO
Cement	Lafarge	St. Constant, QC	I / II	General / Moderate Sulfate	M 85
Fly Ash	Charah	Northbend, OH	F	Low Calcium Fly Ash	M 295
Slag	Lafarge Newcem	Baltimore, MD	120	High Activity Index	M 302

Table 3.3: Chemical admixtures

Material	Manufacturer	Product	Type	Description	AASHTO
Adm. 1	Sika	Sika AEA-14	P-AEA	Air Entraining	M 154
Adm. 2	Sika	Sikament AFM	A	Water Reducing	M 194

Table 3.4: Mix design quantities from batch tickets

Mix No.	3/8 in. (lbs.)	3/4 in. (lbs.)	Fine (lbs.)	Cement (lbs.)	Fly Ash (lbs.)	Slag (lbs.)	Water (gal.)	Adm. 1 (oz.)	Adm. 2 (oz.)
1	356.8	1416.9	1091.9	556.0	98.0	–	33.6	3.0	46.0
2	355.0	1420.0	1095.0	461.0	197.0	–	34.0	3.0	39.5
2B[1]	345.0	1409.8	1086.4	328.0	196.0	–	31.1	3.8	31.2
3	355.0	1420.1	1099.6	492.0	–	166.0	33.6	2.0	46.2
4	345.0	1393.9	1094.2	328.0	–	330.0	33.1	3.2	39.6
5	697.7	1098.5	1169.3	418.0	180.0	–	30.3	4.8	60.0
5B[2]	697.7	1098.5	1169.3	418.0	180.0	–	31.6	4.8	60.0
5C[3]	697.7	1098.5	1169.3	418.0	180.0	–	32.0	4.8	60.0
6	682.1	1085.0	1164.0	300.0	–	300.0	31.7	3.0	42.7

Notes:
[1] Mix design formulation was originally designed with the quantities identified for Mix No. 2. However, a batching error occurred during production, which resulted in the modified mix design formulation Mix No. 2B. Therefore, the quantities identified for Mix No. 2B were formulated and placed instead of the original Mix No. 2 formulation.
[2] Mix Design Formulation No. 5 was subjected to two instances of water additions onsite. Mix Design Formulation No. 5B is identified as the first instance of water addition onsite.
[3] Mix Design Formulation No. 5 was subjected to two instances of water additions onsite. Mix Design Formulation No. 5C is identified as the second instance of water addition onsite.

3.1 Combined Aggregate System

The combined aggregate system of each mix design formulation is calculated based on the *AASHTO T 27 Sieve Analysis of Fine and Coarse Aggregates* test results and the aggregate quantities identified (Table 3.5). The nominal maximum aggregate size (NMAS) for each mix design formulation is 3/4 in.

Table 3.5: Combined aggregate system particle size distribution

Test Method	Property		Mix 1	Mix 2B	Mix 3	Mix 4	Mix 5[1]	Mix 6
T 27	Percentage by Mass Passing (%)	1-1/2 in.	100.0	100.0	100.0	100.0	100.0	100.0
		1 in.	100.0	100.0	100.0	100.0	100.0	100.0
		3/4 in.	97.0	97.0	97.0	97.0	97.8	97.8
		1/2 in.	71.3	71.2	71.3	71.5	78.5	78.5
		3/8 in.	66.1	66.0	66.2	66.3	73.2	73.2
		No. 4	45.3	45.4	45.4	45.7	48.3	48.5
		No. 8	37.9	38.0	38.0	38.4	39.7	39.9
		No. 16	27.0	27.1	27.1	27.4	28.3	28.5
		No. 30	16.4	16.4	16.4	16.6	17.0	17.1
		No. 50	6.1	6.1	6.1	6.2	6.3	6.4
		No. 100	1.9	1.9	1.9	1.9	2.0	2.0
		No. 200	0.5	0.5	0.5	0.5	0.5	0.5

Notes:
[1] Mix Design Formulation No. 5B and No. 5C contained the same combined aggregate system particle size distribution as Mix Design Formulation No. 5.

The combined aggregate system for each mix design formulation is then analyzed with the optimization tools identified herein, including the Tarantula Curve, Shilstone Workability-Coarseness Chart *(4),* and void content. Rather than having gap grading, it is desirable to blend different aggregate sizes to obtain a smooth grading curve for the combined aggregates system *(1).* Mixture design spreadsheets were used by the cement concrete producer (Table 2.1) to conduct the tests necessary to show that the aggregate system satisfied the requirements for a smooth aggregate grading curve.

3.1.1 Tarantula Curve

The Tarantula Curve (Table 3.6 and Figure 3.1) identifies the ideal combined aggregate system particle size distribution for a given cement concrete mix design *(1).* The retained ranges in Table 3.6 indicate the desirable range for the combined aggregate particle distribution that will result in good quality concrete. Proper aggregate grading is fundamental, because aggregate distribution influences water demand, workability, and paste contents *(1).* Mix design formulations that contain aggregate gradations within the limits drawn as dashed lines in Figure 3.1 (Tarantula Curve) have been known to exhibit quality performance characteristics in the field, including good workability, surface finishing, and cohesion, as well as resistance to segregation and edge slumping.

Table 3.6: Tarantula Curve particle size distribution

Sieve Opening	Percentage by Mass (%)				
	Passing	Retained	Retained Ranges		
1-1/2 in.	100	0	0	–	–
1 in.	92	8	0–16	–	–
3/4 in.	82	10	0–20	–	–
1/2 in.	69	13	4–20	–	–
3/8 in.	56	13	4–20	–	–
No. 4	43	13	4–20	–	–
No. 8	37	6	0–12	Coarse Sand 20–40	–
No. 16	31	6	0–12		–
No. 30	18	13	4–20		Fine Sand 25–40
No. 50	5	13	4–20	–	
No. 100	0	5	0–10	–	
No. 200	0	0	0–1	–	

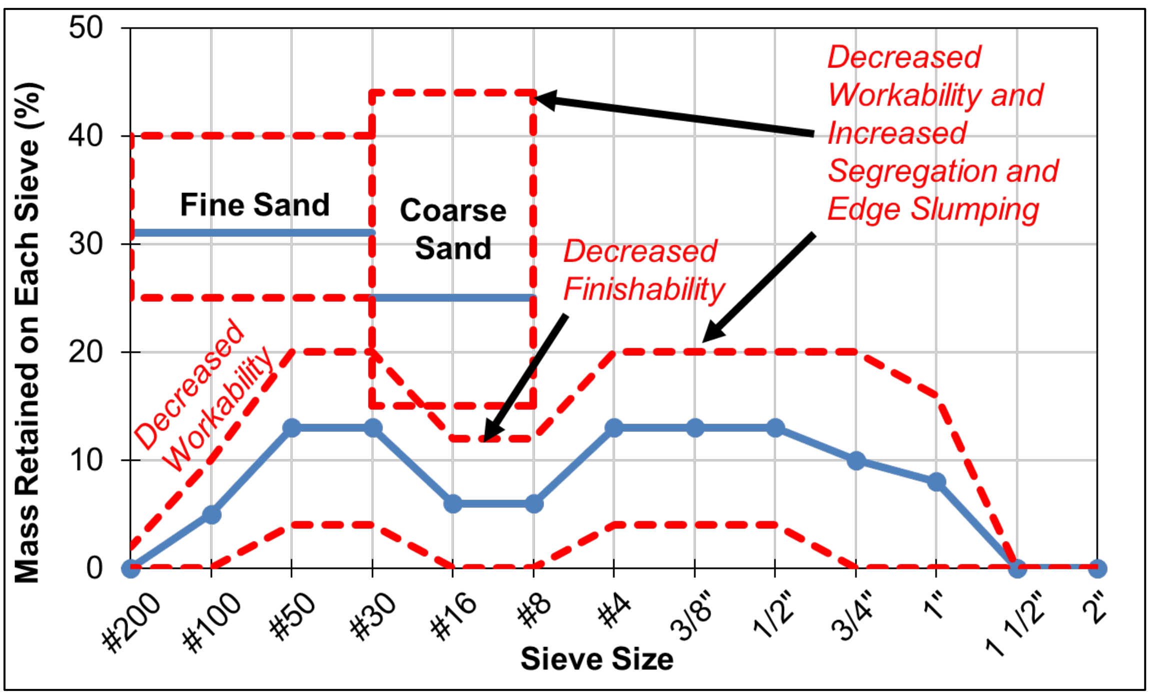


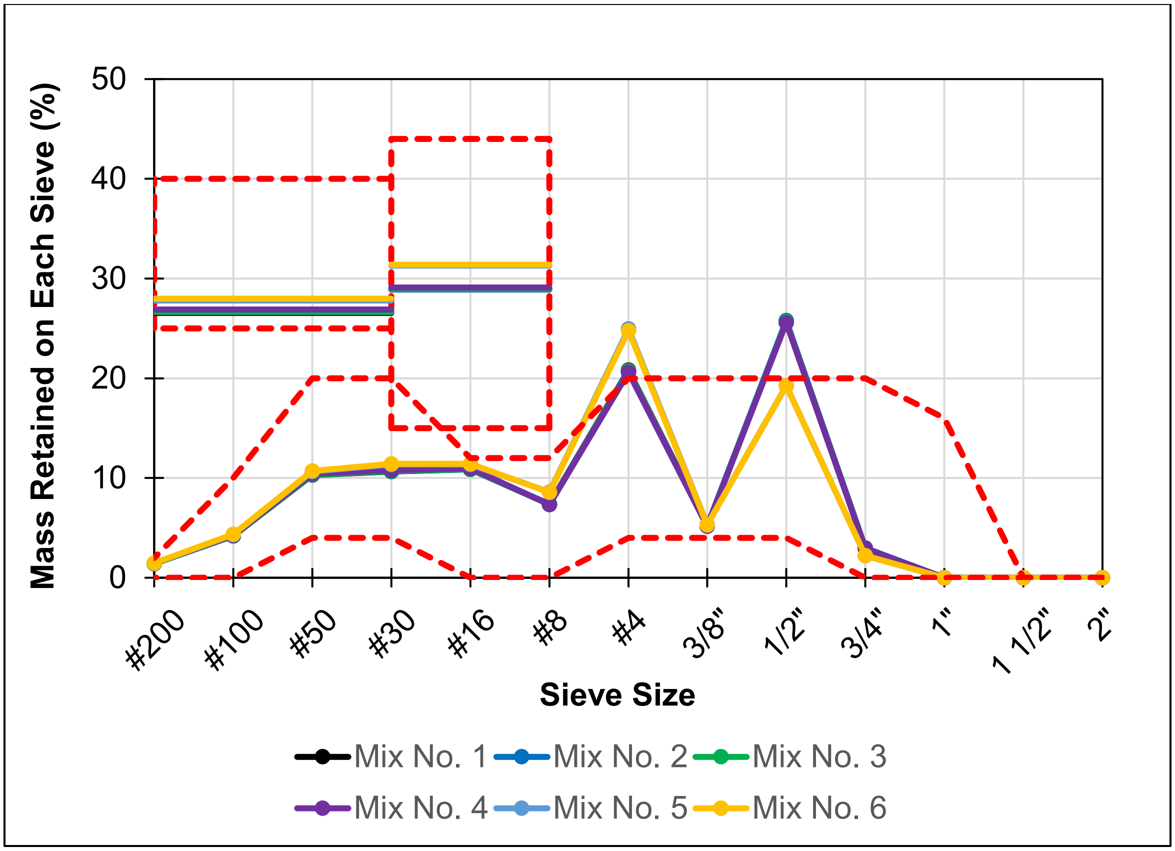
Figure 3.1: Tarantula Curve particle size distribution

The combined aggregate systems for each mix design formulation were compared to the Tarantula Curve in Table 3.7 and Figure 3.2.

Table 3.7: Tarantula Curve results[1]

Test Method	Property		Mix 1	Mix 2B	Mix 3	Mix 4	Mix 5[2]	Mix 6	Criteria		
T 27	Percentage by Mass Retained on Each Sieve (%)	1-1/2 in.	0.0	0.0	0.0	0.0	0.0	0.0	–	–	–
		1 in.	0.0	0.0	0.0	0.0	0.0	0.0	0–16	–	–
		3/4 in.	3.0	3.0	3.0	3.0	2.2	2.2	0–20	–	–
		1/2 in.	**25.7**	**25.8**	**25.7**	**25.6**	19.3	19.2	4–20	–	–
		3/8 in.	5.2	5.2	5.2	5.2	5.3	5.3	4–20	–	–
		No. 4	**20.8**	**20.7**	**20.8**	**20.6**	**24.9**	**24.8**	4–20	–	–
		No. 8	7.4	7.3	7.4	7.3	8.6	8.6	0–12	Coarse Sand 20–40	–
		No. 16	10.9	10.9	10.9	11.0	11.4	11.4	0–12		–
		No. 30	10.7	10.7	10.7	10.8	11.4	11.4	4–20		Fine Sand 25–40
		No. 50	10.3	10.3	10.3	10.4	10.6	10.7	4–20	–	
		No. 100	4.2	4.2	4.2	4.2	4.3	4.4	0–10	–	
		No. 200	1.4	1.4	1.4	1.4	1.5	1.5	0–1	–	

Notes:
[1] Cells in red indicate values that exceed selected criteria.
[2] Mix Design Formulation No. 5B and No. 5C contained the same combined aggregate system particle size distribution as Mix Design Formulation No. 5.



Note: Mixes 1 through 4 are covered by the purple line and Mixes 5 and 6 are covered by the yellow line.

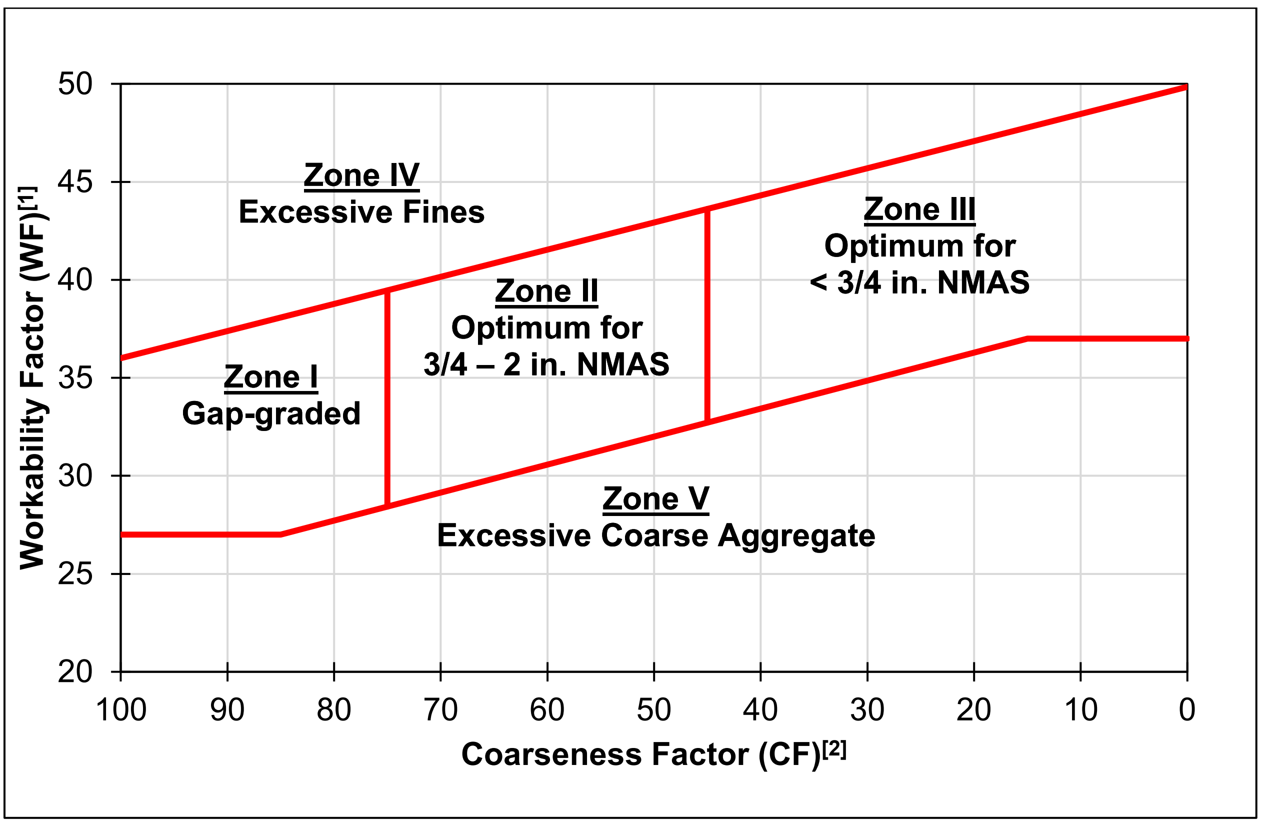
Figure 3.2: Tarantula Curve results

3.1.2 Shilstone Workability-Coarseness Chart

The Shilstone Workability-Coarseness Chart (Figure 3.3) identifies the ideal workability factor and coarseness factor for a given cement concrete mix design *(1,4)*. Mix design formulations that compare favorably to the Shilstone Workability-Coarseness Chart zones have been known to exhibit quality performance characteristics in the field, including quality workability, as well as resistance to segregation during consolidation and finishing, variable strength, shrinkage, cracking, curling, spalling, and scaling.

Table 3.8: Shilstone Workability-Coarseness Zones

Zone	Property	Cause
Zone I	Gap-graded; High potential for segregation during placement and consolidation; Cracking, blistering, spalling, and scaling	Deficiency in intermediate particles; Non-cohesive
Zone II	Optimum mixture for nominal maximum aggregate size from 2 in. to ¾ in.	Optimized workability factor and coarseness factor
Zone III	Optimum mixture for nominal maximum aggregate size < ¾ in.	Optimized workability factor and coarseness factor
Zone IV	Sticky; High potential for segregation during consolidation and finishing; Variable strength, high shrinkage, cracking, curling, spalling, and scaling	Excessive fines
Zone V	Rocky; Lacking plasticity	Excessive amount of coarse and intermediate aggregate



Notes:
[1] The workability factor is determined by the equation WF = W + (2.5(C - 564) / 94), where WF = workability factor, W = percentage passing No. 8 sieve, and C = total cementitious materials content.
[2] The coarseness factor is determined by the equation CF = (Q/R) / 100, where CF = coarseness factor, Q = cumulative percentage retained on 3/8 in. sieve, and R = cumulative percentage retained on No. 8 sieve.

Figure 3.3: Shilstone Workability-Coarseness Chart

The coarseness factor and workability factor for each mix design formulation were calculated in Table 3.9 and compared to the Shilstone Workability-Coarseness Chart Zones in Figure 3.4.

Table 3.9: Shilstone Workability-Coarseness results[1]

Property	Mix 1		Mix 2B		Mix 3		Mix 4		Mix 5[2]		Mix 6		Criteria
CF	54.6	Zone II	54.8	Zone II	54.6	Zone II	54.7	Zone II	**44.6**	**Zone III**	**44.5**	**Zone III**	Zone II
WF	40.3		37.0		40.5		40.9		**40.6**		**40.9**		

Notes:
[1] Cells in red indicate values that exceed selected criteria.
[2] Mix Design Formulation No. 5B and No. 5C contained the same combined aggregate system particle size distribution as Mix Design Formulation No. 5.

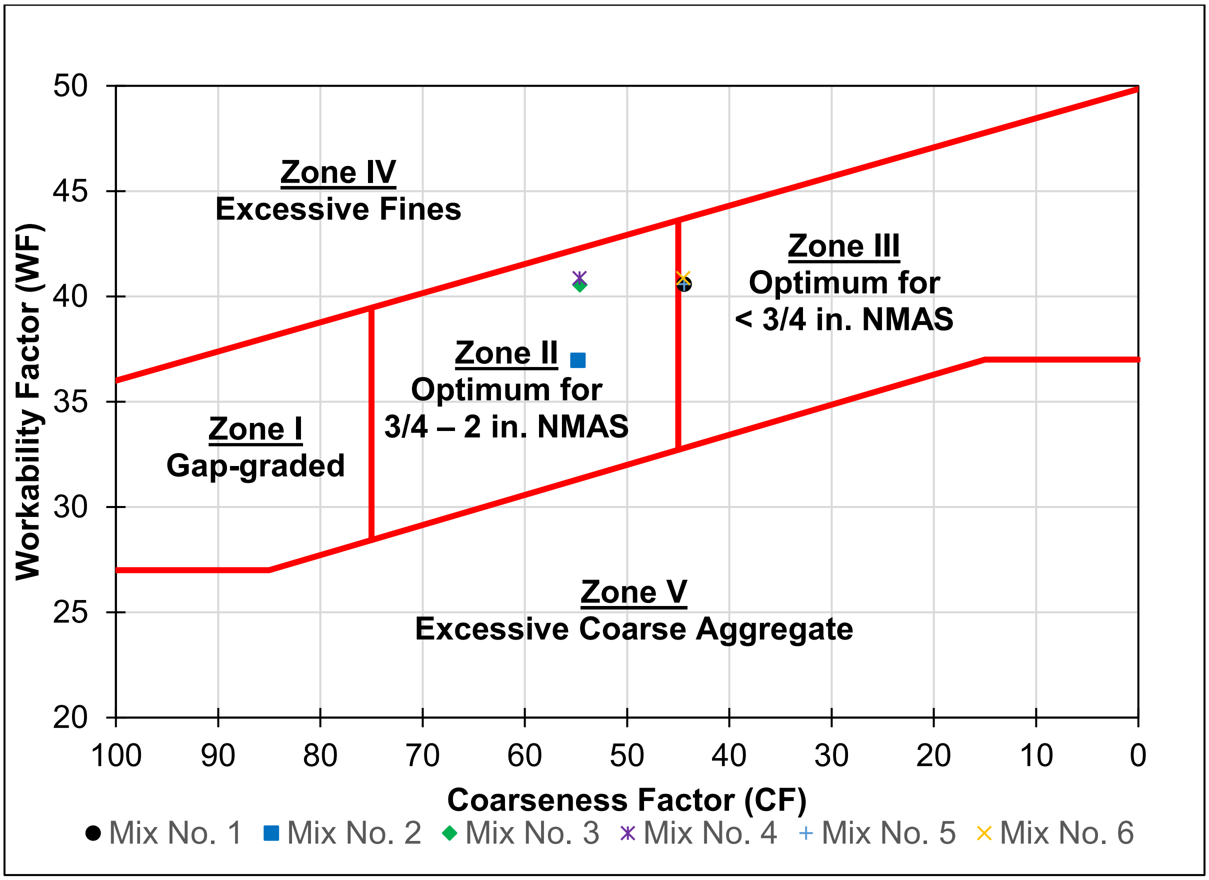


Figure 3.4: Shilstone Workability-Coarseness results

3.1.3 Void Content

The void content for a given mix design formulation estimates the required design proportions of paste (water and cement) needed to adequately fill the voids, to provide sufficient separation between the aggregate particles, promote workability, and promote effective bonding of particles. The void content calculation is also used to calculate the paste content to void content (PC/VC) ratio and excess paste content (EPC) of a given mix design formulation (see Section 3.2.4 for PC/VC and EPC results and analysis).

The void content is determined by the following equations, where W = Weight (lbs.), SG = Specific Gravity, D = Density (pcf), V = Volume (cf), UW = Unit Weight (pcf), and VC = Void Content (%):

$V_{CEMENT} = W_{CEMENT} / (SG_{CEMENT} * D_{WATER})$
$V_{SCM} = W_{SCM} / SG_{SCM} * D_{WATER}$
V_{WATER}[1] $= V_{WATER}$ in gal. / 7.48 gal. per cf
$V_{COARSE} = W_{COARSE} / (SG_{COARSE} * D_{WATER})$
$V_{FINE} = W_{FINE} / (SG_{FINE} * D_{WATER})$
$V_{AIR} = 27$ cf * V_{AIR} in %
V_{YIELD}[2] $= V_{CEMENT} + V_{SCM} + V_{WATER} + V_{COARSE} + V_{FINE} + V_{AIR}$
$VC_{COARSE} = [((SG_{COARSE} * D_{WATER}) – UW_{COARSE}) / (SG_{COARSE} * D_{WATER})]$
$VC_{FINE} = [((SG_{FINE} * D_{WATER}) – UW_{FINE}) / (SG_{FINE} * D_{WATER})]$
$VC_{AGGREGATE} = [(V_{COARSE} / (V_{COARSE} + V_{FINE})) * VC_{COARSE} + (V_{FINE} / (V_{COARSE} + V_{FINE})) * VC_{FINE}]$
$VC_{CONCRETE} = [VC_{AGGREGATE} * ((V_{COARSE} + V_{FINE}) / V_{YIELD})]$

Notes:
[1] The volume of water is defined as the sum of the volume of mixing water, aggregate moisture, and admixture water.
[2] The volumes of the admixtures incorporated into the mix design formulations were determined to be insignificant to the yield, and therefore, the calculated yield does not include the volume of the admixtures incorporated into the mix design.

The aggregate void content for each mix design formulation is reported in Table 3.10.

Table 3.10: Aggregate void content results

Property	Mix 1	Mix 2B	Mix 3	Mix 4	Mix 5[1]	Mix 6	Criteria
Aggregate Void Content (%)	24.2	24.5	24.1	23.8	25.1	25.1	–

Note:
[1] Mix Design Formulation No. 5B and No. 5C contained the same combined aggregate system particle size distribution as Mix Design Formulation No. 5.

3.2 Paste System

The paste system of each mix design formulation was analyzed with the optimization tools identified herein, including water-cementitious ratio, air entrainment, cementitious materials and content, chemical admixtures, and paste content.

3.2.1 Water-Cementitious Ratio

The water-cementitious (w/cm) ratio for a given mix design formulation is directly correlated to the field performance. Cement concrete sidewalk mix design formulations that compare favorably to the American Concrete Institute *ACI 201.2R* water-cementitious ratio criteria (Table 3.11) are recommended, because they exhibit quality performance characteristics in the field, including quality strength and concrete to reinforcement bonding, as well as resistance to freezing, thawing, deicing, sulfate reaction, corrosion of steel reinforcement, drying shrinkage, cracking, and volume change from wetting and drying *(5)*. For plain concrete, *ACI 318-19* recommends less than or equal to 0.45 w/cm ratio also *(6)*. The water-cementitious ratio is determined by calculating the total water content by mass and dividing by the total cement and SCM content by mass.

Table 3.11: Water-cementitious ratio for freezing, thawing, and deicing resistance

Exposure Class	Severity	Condition	W/cm Ratio Criteria
F3	Very Severe	Exposed to freezing and thawing cycles and accumulation of snow, ice, and deicing chemicals; Frequent exposure to water	≤ 0.45[1]

Note:
[1] *ACI 318-19* §19.3.2.1 recommends less than or equal to 0.40 w/cm for reinforced concrete and 0.45 w/cm for plain concrete. A minimum concrete compressive strength of 4500 psi is specified for plain concrete *(6)*.

The water-cementitious ratio for each mix design formulation is compared to the water-cementitious ratio criteria in Table 3.12 for plain concrete.

Table 3.12: Water-cementitious material ratio results[1]

Property	Mix 1	Mix 2B	Mix 3	Mix 4	Mix 5	Mix 5B	Mix 5C	Mix 6	Criteria
Freezing, Thawing, and Deicing Resistance, w/cm	0.43	**0.49**	0.43	0.42	0.42	0.44	0.45	0.44	≤ 0.45

Note:
[1] Cells in red indicate values that exceed selected criteria.

3.2.2 Supplementary Cementitious Materials Content

Supplementary cementitious materials (SCM) incorporated into mix design formulations promote quality properties of the cement concrete. However, incorporation of supplementary cementitious materials in cement concrete may affect the properties of both fresh (Table 3.13) and hardened concrete (Table 3.14). These tables were adapted from Taylor et al. *(1)*. Ample communication between the cement concrete producer and the contractor is required so that adequate adjustments in workmanship practices, including placement, finishing, and curing, and other practices can be made to account for these changes in properties, to prevent scaling. Increased scaling can be attributed to the inadequate workmanship practices that result from overlooking the changes in concrete properties identified in Table 3.13. Adequate adjustments in workmanship practices, including placement, finishing, and curing, and other practices can be made to account for these changes in properties, to prevent scaling.

Table 3.13: Effects on properties of fresh concrete[1]

Property	Fly Ash (Class F)	Slag
Water Content	↓	↓
Workability	↑	↑
Segregation	↓	↕
Bleeding	↓	↕
Setting Time	↑	↑
Air Content	↓	↔
Heat of Hydration	↓	↓

Legend:
↑ = Increases; ↓ = Decreases; ↕ = Increases or Decreases; ↔ = No Impact; ↑ ↔ = Increases or No Impact

Note:
[1] Adapted from Taylor et al., 2007 *(1)*.

Table 3.14: Effects on properties of hardened concrete[1]

Property	Fly Ash (Class F)	Slag
Curing Duration	↑	↑
Early Age Strength	↓	↕
Long Term Strength	↑	↑
Abrasion Resistance	↔	↔
Drying Shrinkage and Creep	↔	↔
Permeability	↓	↓
Corrosion	↓	↓
Alkali Silica Reaction	↓	↓
Sulfate Reaction	↓	↓
Freezing and Thawing	↔	↔
Scaling[2]	↑ ↔	↑ ↔

Legend:
↑ = Increases; ↓ = Decreases; ↕ = Increases or Decreases; ↔ = No Impact; ↑ ↔ = Increases or No Impact

Note:
[1] Adapted from Taylor et al., 2007 *(1)*.
[2] SCMs incorporated into mix designs can delay maturity development and influence properties of fresh concrete. Increased scaling can be attributed to the inadequate workmanship practices that result from overlooking the changes in concrete properties identified in Table 3.13. Adequate adjustments in workmanship practices, including placement, finishing, and curing, and other practices can be made to account for these changes in properties, to prevent scaling.

Cement concrete sidewalk mix design formulations exceeding the maximum allowable American Concrete Institute *ACI 201.2R (5)* SCM content criteria (Table 3.15) may result in decreased resistance to freezing, thawing, deicing cycles, due to the reasons identified herein.

Table 3.15: SCM content for freezing, thawing, and deicing resistance

Exposure Class	Severity	Condition	Material	Replacement by Weight of Cement (%) Criteria
F3	Very Severe	Exposed to freezing and thawing cycles and accumulation of snow, ice, and deicing chemicals; Frequent exposure to water	Low Calcium Fly Ash (F) Content (< 8% CaO)	≤ 25
			Slag Content	≤ 50

The SCM content for each mix design formulation is compared to the SCM content criteria in Table 3.16.

Table 3.16: SCM content results[1]

Property	Mix 1	Mix 2B	Mix 3	Mix 4	Mix 5[2]	Mix 6	Criteria
Fly Ash Content (%)	15.0	**37.4**	–	–	**30.1**	–	≤ 25.0
Slag Content (%)	–	–	25.2	**50.2**	–	50.0	≤ 50.0

Notes:
[1] Cells in red indicate values that exceed selected criteria.
[2] Mix Design Formulation No. 5B and No. 5C contained the same SCM content as Mix Design Formulation No. 5.

3.2.3 Chemical Admixtures

Chemical admixtures are used to enhance cement concrete properties and maintain consistency during mixing, transporting, placing, finishing, and curing. Examples of beneficial chemical admixtures are identified in Table 3.17.

Table 3.17: Chemical admixtures

Spec.	Type	Chemical Admixture	Properties
M 194	A	Water-Reducing	Increases Workability and Air Content; Decreases Water Demand (5%–10%, 3–6 in. Slump)
	B	Retarding	Increases Initial and Final Setting Time, Air Content, Long-Term Strength; Offsetting of Accelerating Effect of Hot Weather; Decreases Early-Age Strength
	C	Accelerating	Increases Early-Age Strength; Decreases Initial and Final Setting Time
	D	Water-Reducing and Retarding	Type A and Type B Admixture Properties
	E	Water-Reducing and Accelerating	Type A and Type C Admixture Properties
	MRWRA	Mid-Range Water-Reducing	Type A and Type F Admixture Properties; Increases Workability (Especially Concrete with SCMs); Decreases Water Demand (6%–12%, 5–8 in. Slump)
	F	High-Range Water-Reducing	Increases Workability (More Effective than Type A), Air Content, Early-Age Strength, and Ultimate Strength; Decreases Water Demand (12%– 40%, > 6 in. Slump) and Permeability
	G	High-Range Water-Reducing and Retarding	Type F and Type B Admixture Properties
	S-SRA	Shrinkage Reducing	Increases Setting Time; Decreases Drying Shrinkage Cracking and Bleed Rate
	S-CRA	Crack Reducing	Decreases Cracking (More Effective than SRAs) and Crack Width
M 154	P-AEA	Air-Entraining	Increases Cohesion, Workability, Stabilization of Air Bubbles, Resistance to Freezing, Thawing, and Deicing, Resistance to Alkali-Reactive Environment, and Resistance to Sulfate Reaction
C1622	CWA	Cold Weather	Increases Hydration Rate; Decreases Freezing Point of Mixing Water

Each mix design formulation incorporated Type A water-reducing chemical admixture (see Section 3.0 for complete mix design formulations).

3.2.4 Paste Content

The paste content (PC) incorporated into mix design formulations promotes quality properties of the cement concrete, including workability, strength, permeability, and resistance to drying shrinkage and cracking and volume change from wetting and drying.

The cracking tendency of concrete has been known to be significantly reduced when the paste content by volume is less than or equal to 28% (for slip formed concrete pavements, less than or equal to 25%), per *AASHTO PP 84-17 (7).* The paste content by volume of cement concrete is determined by the following equations, where V = Volume (cf) and PC = Paste Content (%):

$$V_P = V_{CEMENT} + V_{SCM} + V_{WATER}$$
$$PC = V_P / V_T$$

The volume of paste should adequately fill the voids and provide sufficient separation between the aggregate particles to promote workability and effective bonding of particles. The paste content to void content (PC/VC) ratio and extra paste content (EPC) are used to evaluate the paste system's ability to provide those properties. Workability increases as the paste content to aggregate void content ratio increases. Decreased paste content to aggregate void content ratios will result in decreased workability, where water-reducing admixtures provide no benefit. The PC/VC ratio is determined by the following equations, where D = Density (pcf), SG = Specific Gravity, UW = Unit Weight (pcf), VC = Void Content (%), V = Volume (cf), PC = Paste Content (%), and PC/VC = Paste Content to Aggregate Void Content Ratio:

$$VC_{COARSE} = [((SG_{COARSE} * D_{WATER}) - UW_{COARSE}) / (SG_{COARSE} * D_{WATER})]$$
$$VC_{FINE} = [((SG_{FINE} * D_{WATER}) - UW_{FINE}) / (SG_{FINE} * D_{WATER})]$$
$$VC_{AGGREGATE} = [(V_{COARSE} / (V_{COARSE} + V_{FINE})) * VC_{COARSE} + (V_{FINE} / (V_{COARSE} + V_{FINE})) * VC_{FINE}]$$
$$VC_T = [VC_{AGGREGATE} * ((V_{COARSE} + V_{FINE}) / V_T)]$$
$$PC/VC = PC / VC_T$$

The excess paste content for workability is determined by the following equation, where PC = Paste Content (%), AC = Air Content (%), VC = Void Content (%), and EPC = Excess Paste Content for Workability (%):

$$EPC = PC + AC - VC_T$$

The paste content for each mix design formulation is compared to the criteria in Table 3.18.

Table 3.18: Paste content results[1]

Property	Mix 1	Mix 2B	Mix 3	Mix 4	Mix 5	Mix 5B	Mix 5C	Mix 6	Criteria
Paste Content (%)	**29.7**	27.0	**29.6**	**29.7**	27.3	**28.1**	**28.2**	27.9	≤ 28.0[2]
Paste Content to Void Content Ratio (PC/VC)	**1.1**	**1.0**	**1.1**	**1.1**	**1.0**	**1.0**	**1.0**	**1.0**	1.25 – 1.75[3]
Excess Paste Content (%)	8.3	6.6	8.6	9.7	4.7	4.4	4.6	6.1	–

Notes:
[1] Cells in red indicate values that exceed selected criteria.
[2] Criteria from *AASHTO PP84 Standard Practice for Developing Performance Engineered Concrete Pavement Mixtures (7)*.
[3] Criteria from Taylor et al., 2007 *(1)*.

3.3 Air Void System

The air void system of each mix design formulation was analyzed with the optimization tools identified herein, including air content and chemical admixtures.

3.3.1 Air Content

The air content for a given mix design formulation is directly correlated to the field performance. Cement concrete sidewalk mix design formulations that compare favorably to the American Concrete Institute *ACI 201.2R* air content criteria (Table 3.19) have been known to exhibit quality performance characteristics in the field, including resistance to freezing, thawing, deicing, and sulfate reaction *(5)*. Each mix design formulation is designed with 3/4 in. NMAS.

Table 3.19: Air content for freezing, thawing, and deicing resistance

Exposure Class	Severity	Condition	Nominal Maximum Aggregate Size (in.)	ACI Air Content (%)
F3	Very Severe	Exposed to freezing and thawing cycles and accumulation of snow, ice, and deicing chemicals; Frequent exposure to water	3/4	7.0 ± 1.5

The air content design targets for the mix design formulations were not provided by the cement concrete producer. However, *AASHTO T 152 Air Content of Freshly Mixed Concrete by the Pressure Method* was conducted on each mix design formulation. The test results for each mix design formulation are reported and compared to the *ACI 201.2R* air content criteria for 3/4 in. NMAS *(5)* identified in Table 3.4.

3.3.2 Air-Entraining Chemical Admixtures

Air-entraining chemical admixtures incorporated into mix design formulations increase cohesion, workability, stabilization of air bubbles, as well as providing resistance to freezing, thawing, and deicing, alkali-reactive environments, and sulfate reaction.

Each mix design formulation incorporated an air-entraining chemical admixture (Table 3.3 and Table 3.4). In addition to *AASHTO T 152 Air Content of Freshly Mixed Concrete by the Pressure Method* testing, *AASHTO TP 118 Air-Void System of Freshly Mixed Concrete by Sequential Pressure Method* was conducted on each mix design formulation to evaluate the quality of the air void system. The test results for each mix design formulation are reported and compared to the *ACI 201.2R* air content criteria *(5)* in Table 3.18 and *AASHTO PP 84* air void system criteria *(7)* in Section 4.2.1.

4.0 Field Sampling and Testing

The aggregate used in the mix design formulations, fresh concrete properties, and hardened concrete properties were tested as described in this chapter. Testing was conducted by different laboratories, depending on the test. The organizations involved in the tests were MassDOT, UMass Amherst, Wiss, Janney, Elstner Associates, Inc. (WJE), and ATC Group Services, LLC.

4.1 Aggregate

The test methods identified in Table 4.1 were conducted to evaluate the aggregate properties of each mix design formulation placed.

Table 4.1: Aggregate testing

Test Method	Description
T 11	Materials Finer Than 75-µm (No. 200) Sieve in Mineral Aggregates by Washing
T 19	Bulk Density ("Unit Weight") and Voids in Aggregate
T 21	Organic Impurities in Fine Aggregates for Concrete
T 27	Sieve Analysis of Fine and Coarse Aggregates
T 84	Specific Gravity and Absorption of Fine Aggregate
T 85	Specific Gravity and Absorption of Coarse Aggregate
T 96	Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine
T 104	Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate
T 112	Clay Lumps and Friable Particles in Aggregate
T 380	Potential Alkali Reactivity of Aggregates and Effectiveness of ASR Mitigation Measures (Miniature Concrete Prism Test, MCPT)

The test results for each cement concrete mix design formulation were compared to the criteria identified in Table 4.2 and Table 4.3.

Table 4.2: *AASHTO M6* fine aggregate requirements test results

Test Method	Property		Result	Criteria
T 11	Material Finer than No. 200 Sieve		1.7	≤ 2.0
T 19	Unit Weight (lb / ft^3)		95	–
	Aggregate Void Content (%)		35.7	–
T 21	Organic Impurities		1	≤ 3
T 27	Percentage by Mass Passing (%)	3/8 in.	100	100
		No. 4	99	95–100
		No. 8	93	80–100
		No. 16	70	50–85
		No. 30	43	25–60
		No. 50	16	10–30
		No. 100	5	2–10
		No. 200	1.3	0–3.0
	Fineness Modulus (FM)		2.74	2.3–3.1
T 84	Bulk Specific Gravity (Dry)		2.62	–
T 104	Sodium Sulfate Soundness (%)		0.6	≤ 10
T 112	Clay Lumps and Friable Particles (%)		0.0	≤ 3.0
T 380	ASR Expansion (%)	56 Days	0.02	≤ 0.04

Table 4.3: *AASHTO M80* coarse aggregate requirements test results

Test Method	Property		3/8 in. (No. 8)		3/4 in. (No. 67)	
			Result	Criteria	Result	Criteria
T 19	Unit Weight (lb / ft^3)		98	–	100	–
	Aggregate Void Content (%)		38.9	–	37.8	–
T 21	Organic Impurities		–	–	–	–
T 27	Percentage by Mass Passing (%)	1 in.	–	–	100	100
		3/4 in.	–	–	94	90–100
		1/2 in.	100	100	42	–
		3/8 in.	90	85–100	34	20–55
		No. 4	25	10–30	9	0–10
		No. 8	8	0–10	3	0–5
		No. 16	3	0–5	–	–
	Fineness Modulus		5.74	–	6.60	–
T 84 T 85	Bulk Specific Gravity (Dry)		2.66	–	2.67	–
T 96	Abrasion (%)		25	≤ 50	21	≤ 50
T 104	Sodium Sulfate Soundness (%)		0.5	≤ 12	0.4	≤ 12
T 112	Clay Lumps and Friable Particles (%)		< 1.0	≤ 2.0	< 1.0	≤ 2.0
T 380	ASR Expansion (%)	56 Days	0.06[1]	≤ 0.04	0.06[1]	≤ 0.04

Note: Coarse aggregate source is "moderately reactive" as defined by AASHTO T 380. However, the cement concrete mix design formulations are mitigated with supplementary cementitious materials, which will prevent alkali silica reaction (ASR) from occurring.

4.2 Fresh Concrete

The test methods identified in Table 4.4 were conducted to evaluate the fresh properties of each mix design formulation placed.

Table 4.4: Fresh concrete testing

Test Method	Description
T 119	Slump of Portland Cement Concrete
T 121	Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete
T 152	Air Content of Freshly Mixed Concrete by the Pressure Method
T 309	Temperature of Freshly Mixed Portland Cement Concrete
TP 118	Air-Void System of Freshly Mixed Concrete by Sequential Pressure Method

Testing was conducted on each cement concrete mix design formulation (with exception of Mix Design Formulation No. 5B). Test results were compared to the criteria identified in Table 4.5.

Table 4.5: Plastic concrete test results[1]

Test Method	Property	Mix 1	Mix 2B	Mix 3	Mix 4	Mix 5	Mix 5C	Mix 6	Criteria
T 119	Slump (in.)	6	8	4.5	5.5	3	2.5	3	–
T 121	Unit Weight (lbs / ft^3)	144.2	139.8	142.6	139.8	143.4	145.2	142.7	–
T 152	Air Content (%)	5.1	6.6	5.5	6.1	5.0	4.0	5.4	7.0 ± 1.5
T 309	Concrete Temp. (°F)	63	59	62	70	60	60	67	50–90
TP 118	Air Void System No.	0.24	0.24	0.23	0.19	0.23	– [2]	0.25	≤ 0.25

Notes:
[1] Cells in red indicate values that exceed selected criteria.
[2] Testing was not conducted.

4.3 Hardened Concrete

The test methods identified in Table 4.6 were conducted to evaluate the hardened concrete properties of each mix design formulation placed. At the time of testing, *ASTM C672 (WITHDRAWN 2021)* was an ASTM standard test method *(8)*. However, the ASTM standard test method was withdrawn in early 2021, after testing was completed. *ASTM C672 (WITHDRAWN 2021)* has been reported as overly aggressive on mix designs containing supplementary cementitious materials. Mixes with supplementary cementitious materials set later and develop strength properties at a lower rate than cement-based mixes, neither of which is addressed in the finishing and curing procedures of *ASTM C672 (WITHDRAWN 2021) (8)*. Therefore, the results from this test method are not to be used as an absolute measure of scaling resistance as the test method is no longer an ASTM standard and does not correlate well to field performance *(8)*.

Table 4.6: Hardened concrete testing

Test Method	Description
T 22	Compressive Strength of Cylindrical Concrete Specimens
T 358	Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration
T 161 (A)	Resistance of Concrete to Rapid Freezing and Thawing
C672[1, 2]	Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals (Withdrawn in 2021)
C856[3]	Petrographic Examination of Hardened Concrete
C457[3]	Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete

Notes:
[1] Tests conducted by MassDOT and UMass Amherst.
[2] Withdrawn by ASTM in 2021 after completion of testing in this research *(8)*.
[3] Tests conducted by WJE.

Testing was conducted on each cement concrete mix design formulation (with exception of Mix Design Formulation No. 5B and No. 5C). Test results were compared to the criteria identified in Table 4.7. Mixes 5B and 5C consist of concrete after additional water was added to the mix. Samples were prepared prior to field water addition.

Table 4.7: Hardened concrete test results[1]

Test Method	Property		Mix 1	Mix 2B	Mix 3	Mix 4	Mix 5	Mix 6	Criteria
T 22	Compressive Strength (psi)	7 Days	3460	**1460**	3860	2890	**2470**	3350	≥ 2800
		28 Days	4160	**2230**	5030	4850	**3510**	5540	≥ 4000
		56 Days	4960	**2770**	5340	5500	4240	6300	≥ 4000
T 358	Chloride Ion Penetration (kΩ-cm)	7 Days	5.9	3.9	8.9	6.6	5.5	7.5	–
		28 Days	**9.2**	**9.9**	**17.1**	**18.6**	**10.1**	**20.0**	≥ 21.0
		56 Days	**14.3**	**18.9**	21.4	26.4	**20.1**	29.7	≥ 21.0
T 161 (A)	Rapid Freezing and Thawing Resistance (Durability Factor)		101	91	101	101	101	103	≥ 80
C672[3]	Scaling Resistance: Standard Moist Cure (Rating)[2][4]		**3.5**	**5.0**	**2.5**	**5.0**	**5.0**	**5.0**	≤ 2.0
C672[3]	Scaling Resistance: Curing using Saturated Cover (Rating)[2][5]		1.0	**4.5**	2.0	**4.5**	**4.0**	**5.0**	≤ 2.0
	Scaling Resistance: Curing using Sealing and Curing Compound (Rating)[2][6]		**3.0**	**3.5**	2.0	**4.0**	**5.0**	**5.0**	≤ 2.0
	Scaling Resistance: No Curing (Rating)[2][7]		1.0	**4.0**	1.0	**5.0**	**3.0**	**5.0**	≤ 2.0
C856[8]	Water-Cementitious Ratio (w/cm)		0.43	0.45	0.43	0.42	0.42	0.41	≤ 0.45
	Paste Content (%)		27.5	**28.3**	**31.5**	**33.4**	**33.6**	**32.2**	≤ 28.0
	Fly Ash Content (%)		**32.5**	**40.0**	–	–	**37.5**	–	≤ 25.0
	Slag Content (%)		–	–	30.0	40.0	–	42.5	≤ 50.0
C457[8]	Air Content (%)		**3.6**	8.2	**4.1**	**4.4**	**4.6**	**4.4**	7.0 ± 1.5
	Spacing Factor (in.)		0.006	0.006	0.005	0.005	0.007	0.006	≤ 0.008
	Specific Surface Area (in^2/in^3)		965	**543**	1039	1067	840	868	≥ 600

Notes:
[1] Cells in red indicate values that exceed selected criteria.
[2] Specimens were rated after 50 freeze-thaw cycles and were based on the rating scale provided in *ASTM C672 (WITHDRAWN 2021) (8)* and identified in Table 4.8. The plotting of the intermediate scaling ratings for each curing method is provided Figure 4.1, Figure 4.2, Figure 4.3, and Figure 4.4.
[3] Withdrawn by ASTM in 2021 *(8)*. This test method has been reported as overly aggressive on mix designs containing supplementary cementitious materials. Mixes with supplementary cementitious materials set later and develop strength properties at a lower rate than cement-based mixes, neither of which is addressed in the finishing and curing procedures of *ASTM C672 (WITHDRAWN 2021)*. Therefore, the results from this test method are not to be used as an absolute measure of scaling resistance as the test method is no longer an ASTM standard and does not correlate well to field performance.
[4] Laboratory moist cure was conducted in accordance with *ASTM C672 (WITHDRAWN 2021)*, which is termed "ideal" curing conditions in this report *(8)*.
[5] Saturated covers were placed onto the specimens in the same manner that the saturated cover cement concrete sidewalk panels were subjected to in order to simulate the same curing conditions.

[6] Liquid membrane-forming compound for curing and sealing was placed onto the specimens in the same manner that the liquid membrane-forming compound for curing and sealing cement concrete sidewalk panels were subjected to in order to simulate the same field conditions.
[7] No curing methods were placed onto the specimens in the same manner that the no curing cement concrete sidewalk panels were subjected to in order to simulate the same field conditions.
[8] Complete WJE report (petrographic analysis and air void system) is contained in Appendix J.

Table 4.8: *ASTM C672 (WITHDRAWN 2021)* rating scale *(8)*

Rating	Condition of Surface
0	No scaling
1	Very slight scaling (3 mm [1/8 in.] depth, max, no coarse aggregate visible)
2	Slight to moderate scaling
3	Moderate scaling (some coarse aggregate visible)
4	Moderate to severe scaling
5	Severe scaling (coarse aggregate visible over entire surface)

4.3.1 *ASTM C672 (WITHDRAWN 2021)* Intermediate Scaling Ratings

A total of 48 samples were tested using *ASTM C672 (WITHDRAWN 2021) (8)*, 12 at the MassDOT Research and Materials (RMS) Laboratory and 36 at UMass Amherst. The specimens were subjected to 50 freeze-thaw cycles or until failure was observed, defined as complete scaling of the surface of the specimens.

Four different curing methods were used to evaluate the scaling performance of the six mix design formulations. The specimens tested at the MassDOT RMS Laboratory were cured in accordance with *ASTM C672 (WITHDRAWN 2021)*. Those tested at UMass Amherst were cured following the same procedures employed in the sidewalks at the test site; they included no deliberate curing (ambient curing), moist curing using saturated covers, or curing using curing and sealing compound.

4.3.1.1 *ASTM C672 (WITHDRAWN 2021) Standard Curing Method*

The *ASTM C672 (WITHDRAWN 2021)* standard specifies that prior to subjecting scaling samples to freeze-thaw cycles, the specimens must be moist-cured by placing in a moist chamber for 14 days after fabricating *(8).* Subsequent to this period, samples must be removed and left to air dry for a period of 14 days at 73.5 ± 3.5°F and 45 to 55 % relative humidity prior to ponding brine on the top surface of the specimens in preparation for freeze-thaw cycling. After brine is ponded, samples are subjected to freeze-thaw cycles and inspected periodically after each five cycles have been conducted.

Scaling results for the MassDOT RMS specimens are presented in Figure 4.1. The plots in the figure are averages of scaling ratings of two specimens per mix reported at 5 to 25 cycle intervals as required by *ASTM C672 (WITHDRAWN 2021) (8)*. As shown, Mixes 1 and 3 were the only mixes that came close to achieving a rating of 2 after 50 freeze-thaw cycles. This target performance, corresponding to slight to moderate scaling, was considered an acceptable limit for this study.

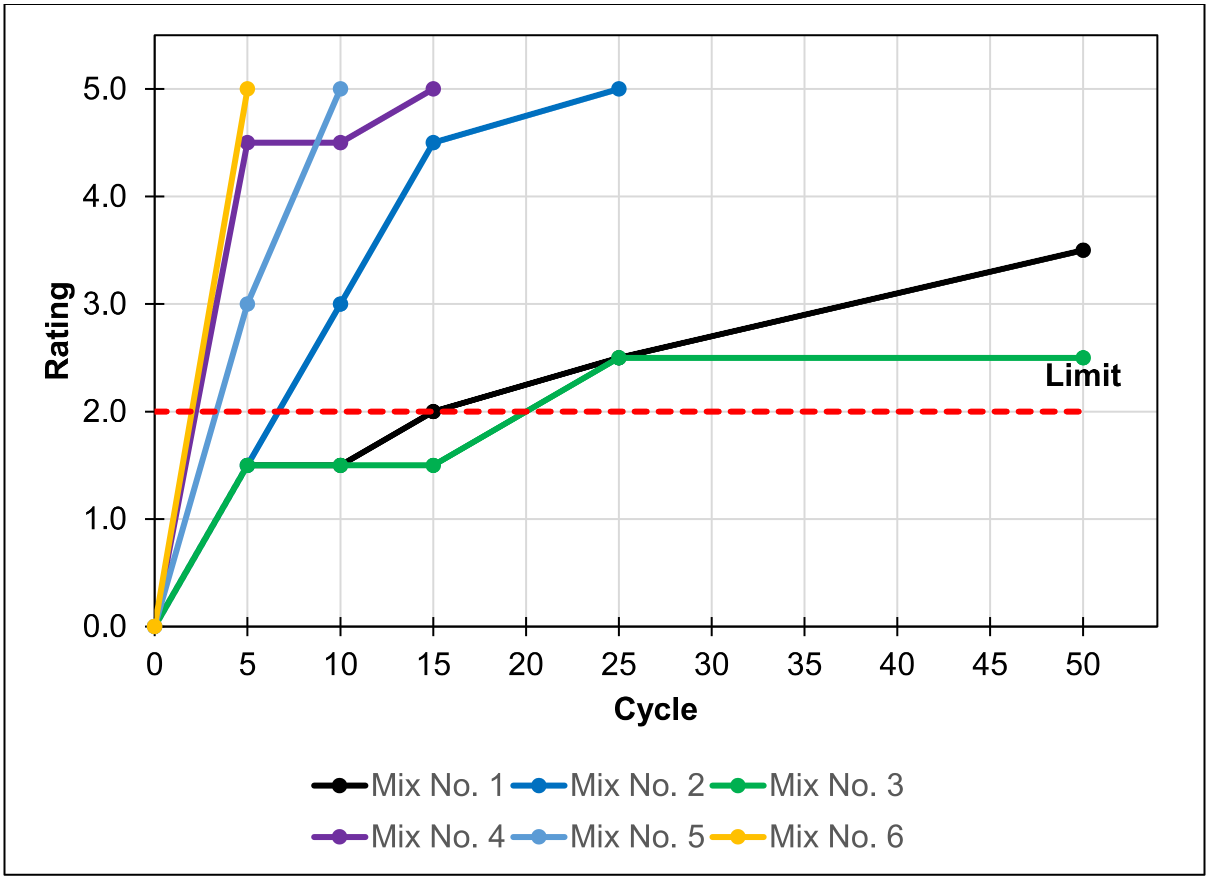


Figure 4.1: ***ASTM C672 (WITHDRAWN 2021)*** **rating versus cycle (standard method)**

4.3.1.2 *UMass Modified Method with Saturated Cover*

The specimens tested at the UMass Amherst laboratory were cured following the three different procedures that were followed in the field in an attempt to replicate field practices as closely as possible. Two replicate specimens were cast for each of the six mixes subjected to the three curing procedures, resulting in a total of 36 specimens tested at UMass Amherst.

Out of the 36 specimens tested at UMass Amherst, 12 specimens were cured using a saturated cover. Full details of the scaling tests conducted at UMass Amherst are included in Appendix F.

Each pair of specimens was placed and cured next to the companion sidewalk panel at the test site. Specimens cured using saturated covers were placed under the same covers as the panels for a period of seven days. Specimens were subsequently transported to Amherst and remained in an outdoor environment for seven days prior to formwork removal. The specimens were moved indoors (laboratory environment with an approximate temperature and humidity compliant with *ASTM C672 (WITHDRAWN 2021) (8)* where dams were fabricated around the perimeter of the top surface to allow ponding the brine solution required to conduct the test. Pictures showing a sample specimen after dams were constructed are shown in Appendix F.

Scaling results for the UMass Amherst specimens cured using a saturated cover are presented in Figure 4.2. The plots in the figure are averages of scaling ratings of two specimens per mix reported at 5 to 25 cycle intervals as required by *ASTM C672 (WITHDRAWN 2021) (8)*. As shown, mixes 1 and 3 were the only mixes that achieved a rating of 2 after 50 freeze-thaw cycles. This target performance, corresponding to slight to moderate scaling, was considered an acceptable limit for this study. Specimens fabricated using mixes 2, 4, 5, and 6 reached scaling ratings of 4 or 5 early during the freeze-thaw cycles.

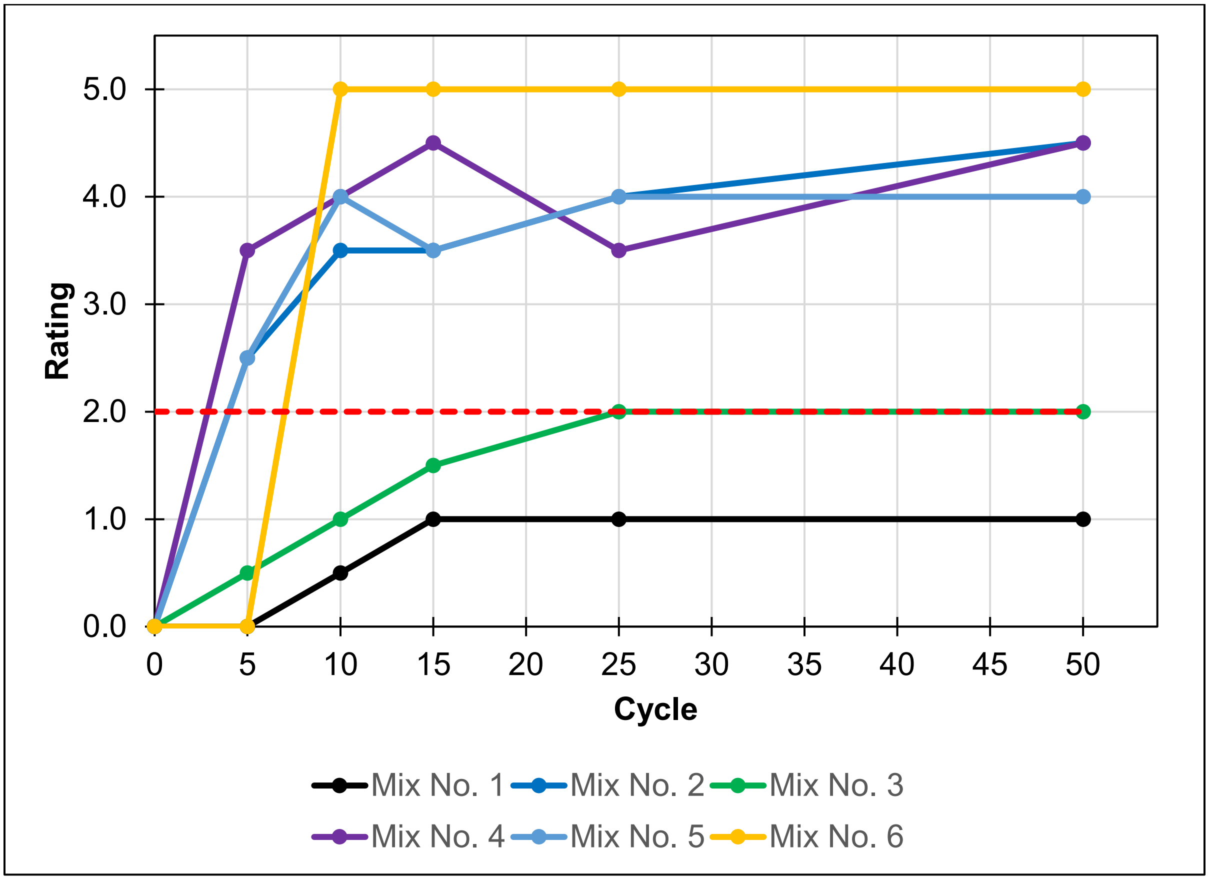


Figure 4.2: *ASTM C672 (WITHDRAWN 2021)* rating versus cycle (modified method saturated cover)

4.3.1.3 *UMass Modified Method with Curing and Sealing Compound*

Out of the 36 specimens tested at UMass Amherst, 12 specimens were cured using a curing and sealing compound. Full details of the scaling tests conducted at UMass Amherst are included in Appendix F.

Each pair of specimens was placed and cured next to the companion sidewalk panel at the test site. For the case of specimens cured using a curing and sealing compound, the compound was applied at the same time as it was being applied to the concrete panels at the test site. Specimens were left at the site for a period of seven days without any cover, after application of the curing compound. Specimens were subsequently transported to Amherst and remained in an outdoor environment for seven days prior to formwork removal. The specimens were moved indoors and placed in a laboratory environment with an approximate temperature and humidity compliant with *ASTM C672 (WITHDRAWN 2021) (8)*. Dams were fabricated around the perimeter of the top surface to allow ponding the brine solution required to conduct the test. Pictures showing a sample specimen after dams were constructed are shown in Appendix F.

Scaling results for the UMass Amherst specimens cured using a curing and sealing compound are presented in Figure 4.3. The plots in the figure are averages of scaling ratings of two specimens per mix reported at 5 to 25 cycle intervals as required by *ASTM C672 (WITHDRAWN 2021) (8)*. As shown, only specimens from Mix 3 reached the target rating of 2 after 50 freeze-thaw cycles and specimens from Mix 1 came close to achieving the target rating. This target performance, corresponding to slight to moderate scaling, was considered an acceptable limit for this study. Specimens fabricated using Mixes 2, 4, 5, and 6 reached scaling ratings of 4 or 5 early during the freeze-thaw cycles.

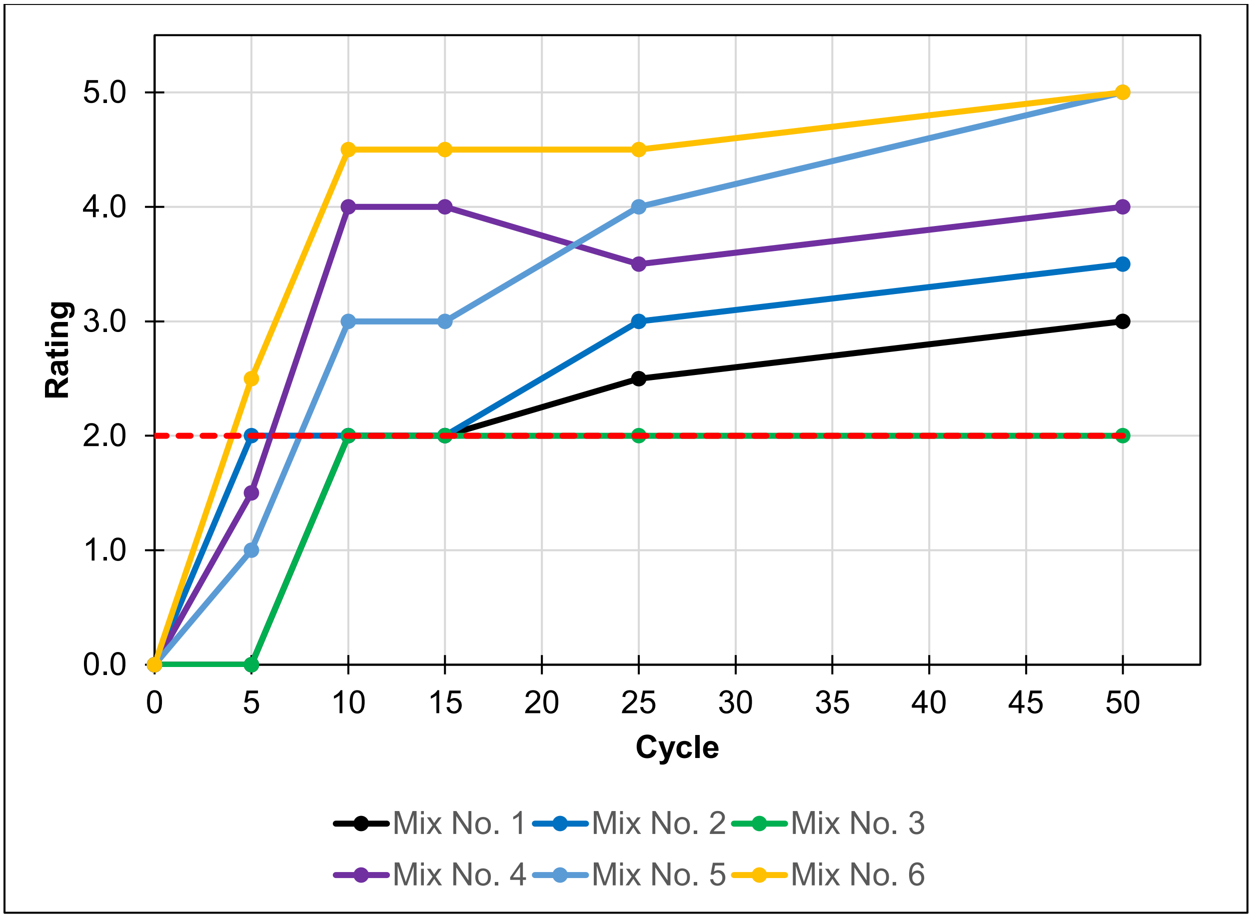


Figure 4.3: *ASTM C672 (WITHDRAWN 2021)* rating versus cycle (modified method curing compound)

4.3.1.4 *UMass Modified Method with No Curing*

Out of the 36 specimens tested at UMass Amherst, 12 specimens were not intentionally cured (no cure). Full details of the scaling tests conducted at UMass Amherst are included in Appendix F.

Each pair of specimens in this group was placed and cured next to the companion sidewalk panel at the test site. The specimens in this group were not subjected to any deliberate curing method; they were left to air dry under environmental conditions encountered at the site. Specimens were left at the site for a period of seven days without any cover. Specimens were subsequently transported to Amherst and remained in an outdoor environment for seven days prior to formwork removal. The specimens were moved indoors and placed in a laboratory environment with an approximate temperature and humidity compliant with *ASTM C672 (WITHDRAWN 2021) (8)*. Dams were fabricated around the perimeter of the top surface to allow ponding the brine solution required to conduct the test. Pictures showing a sample specimen after dams were constructed are shown in Appendix F.

Scaling results for the UMass Amherst specimens without deliberate curing are presented in Figure 4.4. The plots in the figure are averages of scaling ratings of two specimens per mix reported at 5 to 25 cycle intervals as required by *ASTM C672 (WITHDRAWN 2021)*. As shown, specimens from Mixes 1 and 3 remained below the target rating of 2 after 50 freeze-thaw cycle. This target performance, corresponding to slight to moderate scaling, was considered an acceptable limit for this study. Specimens fabricated using Mixes 2, 4, 5, and 6 reached scaling ratings of 4 or 5 early during the freeze-thaw cycles.

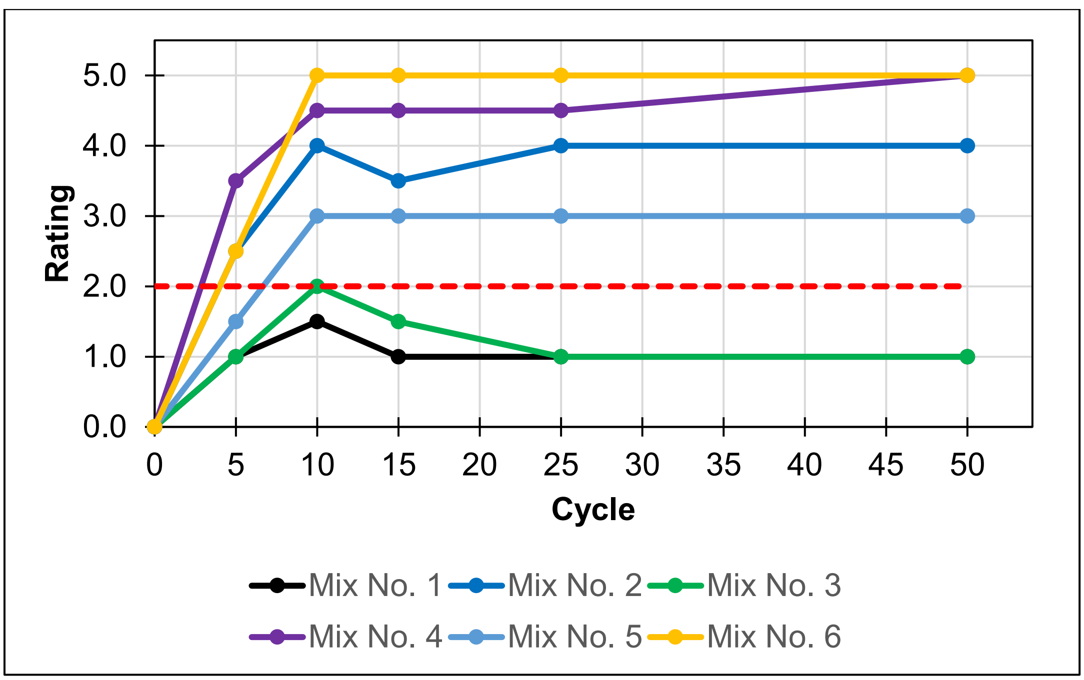


Figure 4.4: *ASTM C672 (WITHDRAWN 2021*) rating versus cycle (modified method no curing)

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5.0 Discussion of Construction Practices at the Test Site

The MassDOT Research and Materials Section and UMass Amherst observed the construction practices performed by the contractor during pre-placement, placement, finishing, and curing in comparison with the requirements identified in Chapter 2.0. Discussion of these comparisons are presented in this chapter.

Average temperatures recorded at the weather station located in the Worcester Regional Airport, MA (42.27N 71.87W, elevation 958 ft.), which is located approximately 15 miles west of the Hopkinton site, were used to identify the environmental conditions from the date of placement through the end of the first winter season after panel placement. Temperature data starting on the first placement day are summarized in Figure 5.1, where the freezing temperature is shown as a horizontal dashed line. The shaded windows marked in the figure indicate potential freezing periods that the sidewalk panels may have experienced during the first fall-winter season after placement. A freezing cycle was not counted if ambient temperature did not remain below freezing for 3 or more consecutive days because of the time lag between concrete freezing and ambient freezing. The first extended period of freezing temperatures started about 5 days after placement and ended approximately 13 days after placement. Therefore, the sidewalk panels experienced their first freeze well before concrete reached its 28-day strength. The plot indicates that the sidewalk panels were subjected to approximately 16 ambient freeze-thaw cycles during the first fall-winter season after placement.

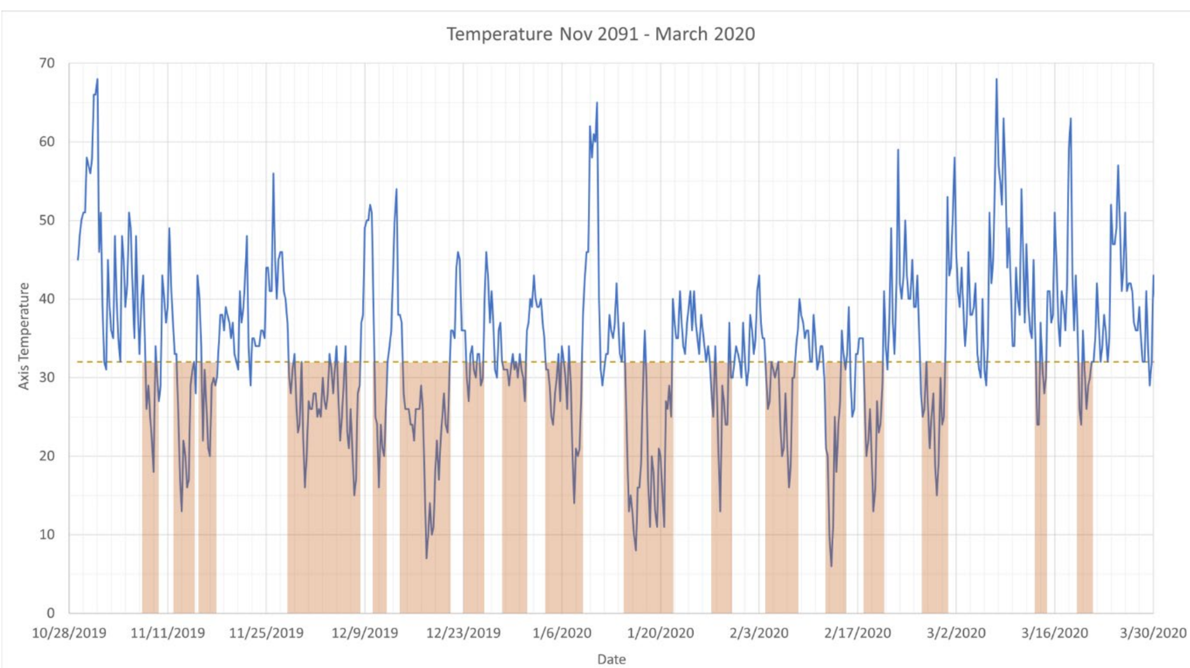

Figure 5.1: Freeze-thaw cycles between late October 2019 and March 2020

5.1 Pre-Placement, Placement, and Finishing Practices

Pre-placement, placement, and finishing practices were to meet the requirements identified in Section 2.2. However, nonconforming placement and finishing practices were observed and are identified in the following subsections herein.

5.1.1 Pre-Placement

Pre-placement activities conformed to the requirements identified in Section 2.2. Figure 5.2 shows site excavation, subgrade compaction, and placement of side forms prior to concrete placement.

Figure 5.2: Compacted subgrade in Panel Groups C, F, I (left); Panel Groups A, D, G, B, E, H (right)

5.1.2 Placement

The first placement occurred on October 29, 2019 (Mixes No. 1, 3, and 5), and the second placement took place on November 5, 2019 (Mixes No. 2, 4, and 6). Placement of panels was staggered, so every other panel was placed during the first placement using wood forms on the sides where pavement or curbing did not contain the concrete (Figure 5.3). A single concrete truck was used to deliver each mix.

Both days were cool and humid. On the second date of placement, a light mist that turned into rainfall occurred during placement. Table 5.1 summarizes temperature and humidity data, along with batching and arrival times of each batched mix.

Table 5.1: Placement weather and timeline

Place-ment	Date	Avg. Temp. (°F)	Relative Humidity (%)	Weather Conditions	Mix No.	Delivery		Placement[1]	
						Batch Time	Arrival Time	Start Time	End Time
1	10/29/2019	55 °F	87%	Overcast; Evening Rain	5	8:41 AM	9:46 AM	9:53 AM	10:32 AM
					3	9:48 AM	10:43 AM	10:45 AM	11:11 AM
					1	10:50 AM	11:40 AM	11:43 AM	12:15 PM
2	11/5/2019	46 °F	77%	Mist; Scattered Rainfall (started at 9:45 AM)	2	7:32 AM	8:25 AM	8:31 AM	8:58 AM
					4	8:11 AM	9:02 AM	9:11 AM	9:37 AM
					6	8:57 AM	9:45 AM	9:55 AM	10:24 AM

Notes:
[1] Panel groups were placed in the following sequence: A, D, G, B, E, H, C, F, I.

Figure 5.3: Concrete placement into form (R); bull floating using magnesium float

The following nonconforming placement practices were observed:

- Water was added to Mix Design Formulation No. 5 and the design's w/cm ratio was exceeded. *AASHTO M 157* requires that the w/cm ratio not be exceeded when water is added on site.
- Water was added on site to Mix Design Formulation No. 5 on two separate occasions. *AASHTO M 157* allows only one instance of water addition onsite.
- Batch tickets do not meet *AASHTO M 157* requirements and quality (see Appendix G for copies of batch tickets).
- The temperature for the second placement fell below 50° F. Below this temperature, cold weather construction practices should be used.

5.1.3 Finishing

Finishing of sidewalks should initiate after bleed water is not observable on the concrete surface. The top surface was thumb-pressed to identify the consistency of the concrete and whether water filled the indentation to identify whether any bleed water was still migrating to the top of panels. Finishing operations began when no water filled the indentation. Information about finishing times and curing times can be found in the field notes that were taken during the two placements, presented in Appendix G. The tools used to finish the sidewalks were in general appropriate, Figure 5.4 (magnesium float and trowels), except for panels in Mix 4 that were intentionally finished using prohibited operations, Figure 5.5 (additional water and steel trowel finish).

The following nonconforming finishing procedures were observed:

- Given the weather conditions (mist, rain), it was difficult to observe whether there was any bleed water on the surface of panels cast during the second placement (Mixes 2, 4, 6). This may constitute a non-conforming finishing operation.
- Water was added to the top surface ("blessing") prior to finishing panels in Mix 4. Note: this was intentionally done to document the performance of poorly finished panels.
- Steel trowels were used to finish panels in Mix 4. Note: this was intentionally done to document the performance of poorly finished panels.

Table 5.2: Finishing

Placement	Date	Avg. Temp. (°F)	Relative Humidity (%)	Weather Conditions	Mix No.	Finishing Initiation after Placing (Hours)
1	10/29/2019	55 °F	87%	Overcast; Evening Rain	5	1.5–2.0
					3	1.5–2.0
					1	1.5–2.0
2	11/5/2019	46 °F	77%	Mist; Scattered Rainfall (started at 9:45 AM)	2	3.0
					4	2.0
					6	1.5–2.0

Figure 5.4: Best practices panels—magnesium float (left) and broom (right)

Figure 5.5: Prohibited practices panels—blessing (left) and steel trowel (right)

5.2 Curing Methods

The following curing methods applied to the panels were to meet the requirements identified in Section 2.3.

- Saturated covers
- Liquid membrane-forming compounds for curing and sealing
- No curing

Nonconforming curing practices for the saturated cover curing and liquid membrane-forming compound for curing and sealing methods were observed and are identified in the following subsections herein. The measured temperature and humidity for the seven days following placement of the panels at the test site are presented in Figures 5.6 and 5.7, respectively. Figure 5.6 also indicates two temperature limits that are critical for proper sidewalk durability performance: 50° F and 32° F. The first temperature is used to limit placement of concrete prior to employing cold weather concreting practices. The second temperature (freezing) indicates is associated with the susceptibility of scaling damage of concrete through freeze-thaw action at its early age. It can be seen that the 50° F limit was violated in both placements and that the 32° F was violated for sidewalks cast in the second placement.

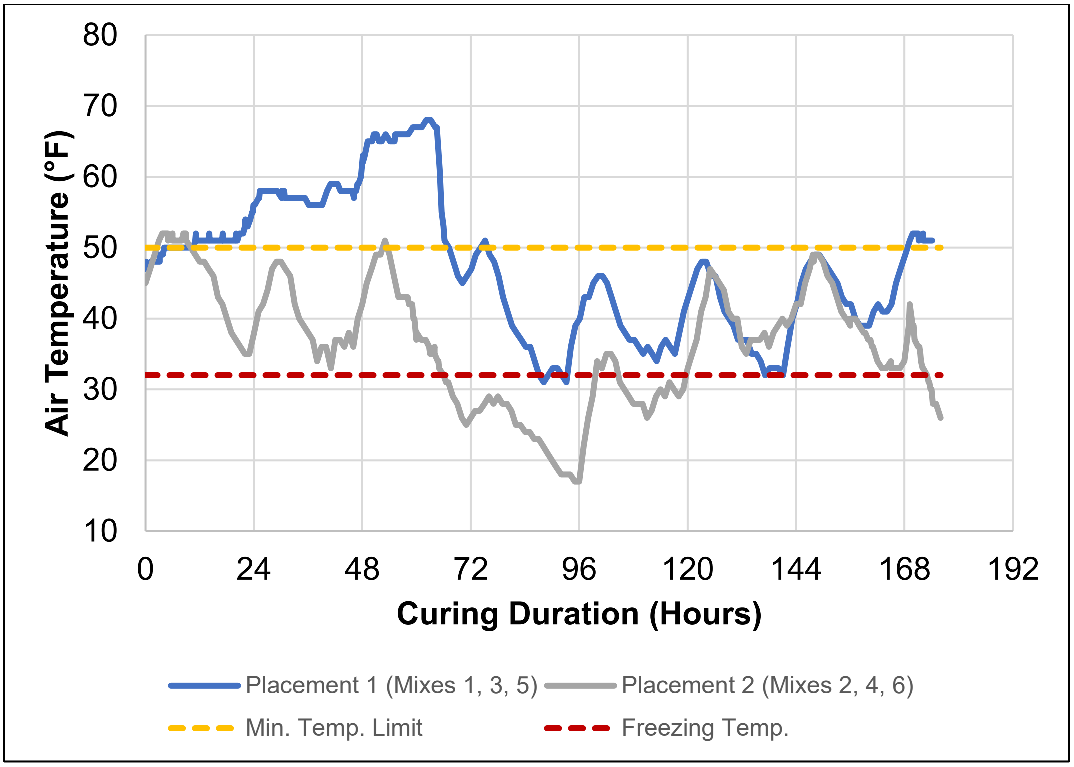

Figure 5.6: Recorded air temperatures during required curing cycle (7 days)

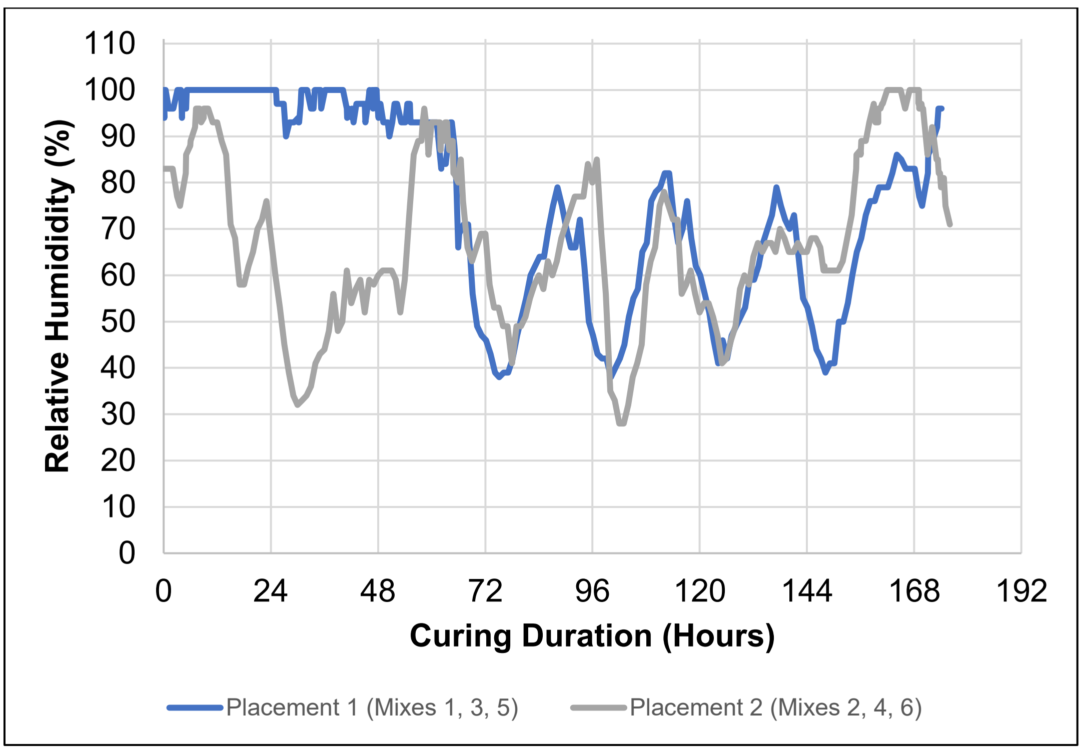

Figure 5.7: Recorded relative humidity during required curing cycle (7 days)

5.2.1 Application of Saturated Cover

Saturated covers (wet burlap) were applied to Panel Groups A, B, and C in Placements 1 and 2 to decrease water evaporation starting at 2:35 p.m. for Placement 1 and 1:05 p.m. for Placement 2 (Figure 5.8). The burlap left noticeable staining on many the moist cure panels, as pictured Figure 5.9, indicating that burlap may not have been properly cleaned (flushed) prior to its use. The saturated covers eventually dried out and were not kept moist throughout the entire duration of the curing cycle (seven days). Cold weather temperature control was not conducted, and the panels were subjected to temperatures below the requirements identified in Section 2.3. The covers were not properly anchored down to the surface, as the wind was able to blow the covers off the surface.

Figure 5.8: Application of saturated cover

Figure 5.9: Staining mark on Panel 1A after burlap removal (November 29, 2019)

The following nonconforming curing practices for the application of saturated cover curing were observed:

- Saturated covers did not contain adequate moisture to prevent moisture loss from the surface of cement concrete and cementitious materials. Saturated covers did not retain sufficient moisture from continuous watering so that a film of water remains on the surface of cement concrete and cementitious materials through the entire duration of the final curing method cycle.
- Saturated covers did not accommodate uniform and slow drying of cement concrete and cementitious materials surfaces immediately prior to removal.
- Saturated covers did not subject all exposed cement concrete surfaces with a continuous application of moisture throughout the entire duration of the final curing cycle and did not provide a controlled and gradual termination of the final curing cycle.
- Improper application of saturated covers and maintenance of moisture in the covers did not allow the concrete near the surface to mature sufficiently to achieve its desired properties, including strength (see point below), permeability, durability, and resistance to freezing, thawing, and deicing cycles, resulting in decreased strength and durability of the top surface of concrete (see Chapter 6.0 for findings from the petrography analyses).
- Minimum sustained concrete temperature (50° F), duration (seven days), and strength (70% f'_c) requirements were not met during the seven-day curing cycle for Mixes 2 and 5 (see Appendix E). Mix 2 only reached an f'c equal to 1460 psi at seven days, and Mix 5 reached an f'c equal to 2470 psi at seven days. The strength after seven days should be at least equal to 2800 psi.
- Cold weather concreting procedures, operations, materials, and equipment required for the curing and protection of the concrete in accordance with *ACI 306.1- 90 Standard Specification for Cold Weather Concreting* were not followed *(9),* despite the low temperatures experienced during placement and within the seven-day curing period.

5.2.2 Application of Liquid Membrane-Forming Compound for Curing and Sealing

A liquid membrane-forming compound for curing and sealing (Cure Shield EX by SpecChem, Appendix B) was applied to the panels identified in Figure 2.1. SpecChem Cure Shield EX is a solvent-based blend of silane and acrylic copolymers designed to cure, seal, harden, and dustproof fresh concrete and provide additional protection to existing concrete. The liquid membrane-forming compound for curing and sealing meets *ASTM C309* and *ASTM C1315* Type I (clear or translucent) Class B (moderate yellowing) curing standards *(10,11).* The manufacturer's instructions called for the compound to be applied as soon as the surface water has disappeared and after the concrete surface will not be marred by the walking applicator. However, it was observed that the compound was applied immediately after the panels were being finished, with the water sheen still present (Figure 5.10). A complete coating of the concrete surface using the curing/sealing compound was applied.

Figure 5.10: Water sheen observed on panel as curing compound is applied

The following are the nonconforming curing practices for the application of liquid membrane-forming compound for curing and sealing:

- *ASTM C1315* Type I (clear or translucent) Class B (moderately yellowing) was applied instead of the required *ASTM C1315* Type I (clear or translucent) Class A (non-yellowing) *(11)*.
- Compound was applied immediately after the panels were finished. Water sheen on the surface of the panels was still present.
- Minimum sustained concrete temperature (50° F), duration (seven days), and strength (70% f'_c) requirements were not met during the curing cycle.
- Cold weather concreting procedures, operations, materials, and equipment required for the curing and protection of the concrete in accordance with *ACI 306.1-90 Standard Specification for Cold Weather Concreting* were not followed *(9),* despite the low temperatures experienced during placement and within the seven-day curing period.

5.2.3 No Curing

No-cure panels from Placement 1 were left to air dry following brooming (Groups G, H, and I). Because mist and scattered rain were falling in Placement 2, no-cure panels were covered under a plastic sheet for approximately 24 hours after brooming to protect them from additional rainfall after placement. These panels were exposed to the air temperature conditions identified in Figure 5.6 and the relative humidity conditions identified in Figure 5.7.

6.0 Petrographic Examination

Wiss, Janey, Elstner Associates, Inc. (WJE) performed petrographic examination on extracted cores from the cement concrete sidewalk panel site. The following commentaries are direct excerpts from their *Petrographic Studies of Concrete Cores* (see Appendix J for full report received from WJE).

WJE performed petrographic studies, air-void system analyses, and chloride ion content determinations for 60 concrete cores extracted from several sidewalk panels adjacent to the MassDOT Materials and Testing Laboratory located in Hopkinton, Massachusetts (test site). The main objective of these tests was to determine the causes of premature scaling observed in the panels, if any. WJE also performed petrographic studies on six cast concrete cylinders sampled during placement of the sidewalk that took place in late October and early November 2019. These tests were conducted in spring 2020 and reported in an interim report dated March 11, 2020.

Sixty cores were taken from the test site and shipped to the WJE laboratory on two dates. WJE completed laboratory studies on the first 24 cores received from the test site in fall 2020 and reported these findings in an interim report dated October 22, 2020. The remaining 36 cores were completed at a later date. WJE completed laboratory analyses of water soluble chloride content for two concrete samples extracted from each of the 60 cores (120 samples total) and issued a memorandum of those test results on December 9, 2020. Chloride contents were determined for slices taken near the top and bottom of the cores received at the WJE laboratory. Full details of the slice location for chloride content analysis are presented in Appendix I.

Sixty 4-inch nominal diameter, full-depth concrete cores were received by the Janney Technical Center of Wiss, Janney, Elstner Associates, Inc. (WJE) in Northbrook, Illinois, on August 4, 2020 (first 24 cores), and September 3, 2020 (remaining 36 cores). The 60 concrete cores represented the six different concrete mixture designs being studied in this project. The mix design information was provided for review, information, and comparison purposes, after examinations of the first 24 cores had been completed. Air contents were not included in the provided mixture designs and were estimated by assuming that the provided quantities of materials yielded 1 cubic yard (27.0 cubic feet) of concrete. The sample ID and corresponding mix design designation for each core are provided in Tables B. 1 through B.12 of Appendix I. Images of the cores, as received in WJE's laboratory, are shown in the datasheets for each core (Appendix F of the WJE report).

The surface condition of the cores generally ranged from an intact broom-finished surface, corresponding to the finishing operation that the sidewalk panels received at the test site, to minor pitting, localized or moderate scaling, or a rough surface exposing abundant aggregate particles. The depth of thescaling observed in the cores received for testing appeared to be generally less than 0.1 inch from the finished surface. The bottom surfaces of the cores were uneven and generally exhibited adhered subbase rock particles, typical of concrete cast against soil. No evidence of vapor retarder installed before the concrete placement was observed. No major cracks were observed in the cores. No steel reinforcement or other embedded items were observed, typical of sidewalk panels.

The concrete represented by the cores was generally well consolidated, with scattered entrapped air voids frequently observed in many cores. Distribution of the material constituents was generally uniform in the body of the concrete below a weak top surface layer, which is described separately below. All cores consisted of the same or similar crushed siliceous rock coarse and fine aggregate dispersed in a generally well air-entrained cementitious paste of either Portland cement and fly ash or Portland cement and slag cement.

Petrographic studies were conducted in accordance with *ASTM C856, Standard Practice for Petrographic Examination of Hardened Concrete*. Photographs were taken of all as-received cores before any sample preparation was conducted (see photos in the datasheets in Appendix F of the WJE report). A slab was cut longitudinally from approximately the middle of each core using a water-cooled, continuous-rim, diamond saw blade. One resulting sawed surface of each slab was lapped using progressively finer diamond-embedded abrasive discs to achieve a fine, matte finish suitable for examination with a stereomicroscope. Lapping exposes textural features such that characteristics of the paste and aggregate can be more easily observed microscopically. Lapped surfaces of the cores can be seen in the datasheets provided in the WJE report for each core. Fresh fracture surfaces were also prepared to study the characteristics of the concrete and for the purposes of measuring carbonation depth from the top surfaces of the cores, when needed, in addition to using thin section observations. A copper probe was used to qualitatively assess paste hardness. A thin section was prepared encompassing the top surface of each core to further assess paste characteristics. The thin sections were examined at magnifications ranging from 3.6X to 630X, using a petrographic (polarized-light) microscope. Point-counts of the cementitious paste constituents were conducted on Core 41 and Core 56.

Hardened air void analyses were conducted in accordance with the modified point-count method (Procedure B) described in *ASTM C457, Standard Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete*. Lapped vertical cross-sections were analyzed at a magnification of 100X. The results of the air void analysis are provided in Tables C.1 through C.6 (Appendix C of the WJE report).

Water-soluble chloride ion concentrations were determined at two depths for each of the concrete core samples. All cores were full-depth cores, measuring between 4-5/8 and 10-1/2 inches in length. Chloride ion concentrations were measured for two 1/4-inch thick slices that were saw cut from each core: one slice sampled between 3/4 and 1 inch from the top surface, and the other slice sampled between 4-1/2 and 4-3/4 inches from the top surface. Each slice was oven-dried and crushed into a fine powder, before being analyzed for water-soluble chloride content in general accordance with *ASTM C1218, Standard Test Method for Water-Soluble Chloride in Mortar and Concrete*.

6.1 Examination of Core Surface

The surface of each extracted core was examined to determine the in-situ properties of each cement concrete panel. The surface examination is vital in determining the quality of the finishing, curing, and protection of the concrete panels, as well as determining the amount of chlorides present from external sources such as deicing agents, all of which are directly related to scaling resistance.

The water-cementitious material ratio results for all 60 cores were greater than the ACI 201.2R recommended ratio of 0.45 (Table 6.1) *(5).* The chloride ion content for the cores extracted from the concrete panels that were subjected to no intentional curing method were 176% higher than the cores corresponding to panels cured via a sealing compound and 239% higher than cores cured using a saturated cover (Table 6.2).

Table 6.1: Water/cementitious ratio results—petrographic analysis of cores[1]

Panel Group	Curing Method	Deicing Method	ACI 201.2R w/cm Ratio Criteria: ≤ 0.45							
			Mix Design Formulation No.							
			1	2B	3	4[2]	5	5B	5C	6
			0.43	0.49	0.43	0.42	0.42	0.44	0.45	0.44
A	Saturated Cover	NaCl	0.58	0.58	0.53	NR[3]	0.53	–	–	0.53
B		NaCl and $MgCl_2$	0.53	0.58	0.53	0.53	–	0.53	–	0.53
C		Residual NaCl from Spreader and Traffic Spray	0.53	0.58	0.53	0.53	–	–	0.53	0.53
									0.53	
D	ASTM C1315 (Curing and Sealing Compound)	NaCl	0.53	0.58	0.53	NR[3]	–	0.53	–	0.53
E		NaCl and $MgCl_2$	0.53	0.58	0.53	0.53	–	–	0.53	0.53
F		Residual NaCl from Spreader and Traffic Spray	0.53	NR[3]	0.53	NR[3]	–	–	≥ 0.50	0.53
						Elev-ated				
G	No Curing	NaCl	0.53	0.58	0.53	0.53	–	0.53	–	0.53
H		NaCl and $MgCl_2$	0.53	0.58	0.53	0.53	–	–	0.53	0.53
I		Residual NaCl from Spreader and Traffic Spray	0.58	0.58	0.53	0.53	–	–	≥ 0.50	0.53
				0.58	0.53				0.53	0.53

Notes:
[1] Cells in red indicate values that exceed selected criteria.
[2] Cement concrete sidewalk panels containing Mix Design Formulation No. 4 were subjected to prohibited placing and finishing practices to study its effect on scaling.
[3] Result was not recorded.

Table 6.2: Chloride ion content results from petrographic analysis of cores

Panel Group	Curing Method	Deicing Method	Chloride Ion Content (ppm)									
			Mix Design Formulation No.								Average	
			1	2B	3	4[1]	5	5B	5C	6	Deicing Method	Curing Method
A	Saturated Cover	NaCl	60	60	50	60	250	–	–	60	90	56
B		NaCl and $MgCl_2$	70	30	25	25	–	100	–	25	46	
C		Residual NaCl from Spreader and Traffic Spray	25	50	25	25	–	–	70 25	25	35	
D	ASTM C1315 (Curing and Sealing Compound)	NaCl	170	50	40	50	–	270	–	40	103	76
E		NaCl and $MgCl_2$	40	25	25	70	–	–	120	25	51	
F		Residual NaCl from Spreader and Traffic Spray	25	70	25	40 25	–	–	310	25	74	
G	No Curing	NaCl	90	30	25	30	–	250	–	25	75	134
H		NaCl and $MgCl_2$	25	25	25	25	–	–	240	25	61	
I		Residual NaCl from Spreader and Traffic Spray	430	380 100	25 40	70	–	–	530 450	50 50	213	

Notes:
[1] Cement concrete sidewalk panels containing Mix Design Formulation No. 4 were subjected to prohibited placing and finishing practices to study its effect on scaling.

The WJE report notes that:

> The surface distress observed in the cores was visible as minor pitting, minor to moderate scaling, and full loss of the broom-finished surface [individual datasheets for each core are included in the appendix containing the full WJE report]. Some surface loss appeared to occur above near-surface aggregate particles, consistent with mortar flaking or pop-offs (as denoted by some engineers and petrographers). Distress consistent with popouts, which are related to unsound aggregate particles, was minimal or not observed. Surface loss over large areas, referred to as sheeted scaling or delamination, was not observed. Delamination frequently occurs to air-entrained concrete subjected to hard-trowel finishing. Trowel-finishing operations expel water and air out of the near-surface concrete, resulting in formation of a densified top layer that is significantly

> different from the concrete immediately under the layer and the bulk concrete at greater depth. Since the finishing may also trap water and air below the densified top layer, a zone may form with greater w/c, greater air content, and therefore a weaker paste that is prone to delaminating at trafficking or loading (even without cyclic freezing-thawing). The subject sidewalk concrete typically received a broom finish, and the weak layer generally occurred uppermost on the finished concrete.

The WJE report further notes that:

> Near-surface conditions were present that would make concrete near the surface of the cores susceptible to premature freeze-thaw distress and general weathering/erosion. Almost all of the 60 cores exhibited a thin, weak, absorptive layer near the top surface of the concrete. The thickness of the weak porous surface mortar ranged from 0.02 to 0.1 inch. The physical properties (lighter gray color, soft to moderately soft paste, dusting, high absorption, suspected bleed channels, early-age drying shrinkage cracks, and carbonation) and the microscopical characteristics of the paste in the weak surface layer appeared to be generally consistent with elevated w/cm and possibly inadequate or less-than-optimal curing. In addition to elevated w/cm and potentially inadequate curing, a thin, weak top layer present in almost all cores appeared to frequently contain fewer air voids than the body concrete. The top surface layer, while thin, did not appear to meet the requirements for freeze-thaw durable concrete.

These observations shed light into the condition of the top surface of the sidewalk panels that might lead to early onset surface scaling from freeze-thaw action.

The importance of proper curing, particularly for cement concrete mixtures containing fly ash and slag, is emphasized in the WJE report as can be seen from the following statements:

> It is more difficult to judge the proper time of finishing for concrete containing SCMs. Concrete containing SCMs also generally requires particular attention to curing to promote conditions needed for the slag hydration or fly ash pozzolanic reactions to proceed. Specifically, prolonged moist curing is often needed for proper resistance to freezing and thawing, especially in the presence of deicing salts. The reported premature surface deterioration is likely due to this weak surface layer. A sealer or a curing compound was observed on the top surface of many cores.

Concrete mixture design did not appear to independently influence the tendency of the panels to suffer surface scaling. The WJE report indicates that “scaling was observed in cores of each mix design. The Mix 3 group cores exhibited no scaling or other form of surface distress, except for Core 54 which contained an overall intact broom-finished surface with minor scaling. Cores of Mix 1 also appeared to be less susceptible to surface distress. Both Mix 1 and Mix 3 cores contain the lowest amounts of SCMs in their respective binary cementitious mixtures. The potential correlation appeared to be true for both sets of cores examined." The petrographic analysis results seem to indicate that Mixes 1 and 3 performed better against scaling. These results should be viewed with caution, however, because some cores were intentionally taken in areas where scaling had been observed in the field, primarily in panels of groups C, F, and I. Winter deicing practices also influenced the occurrence of field scaling.

6.2 Examination of Core Body

The body of each extracted core was examined by the WJE tests to determine the in-situ properties of each cement concrete panel. The body examination is vital in determining the quality of the constituent materials and placement, the structural integrity of the cement concrete panels, and conformance to mix design formulations.

The SCM content, water-cementitious ratio, paste content, air content, air void system spacing factor, and air void system specific surface area results for each cement concrete mix design formulation were compared to the criteria identified in Table 6.3, Table 6.4, Table 6.5, Table 6.6, Table 6.7, and Table 6.8, respectively. The entire WJE petrographic report containing all the results and commentary is located in Appendix I.

Table 6.3: SCM content results from petrographic analysis of cores

Panel Group	Curing Method	Deicing Method	*ACI 201.2R* SCM Content (%) Criteria: ≤ 25.0 Fly Ash, ≤ 50.0 Slag[1]							
			Mix Design Formulation No.							
			1	2B	3	4[1]	5	5B	5C	6
			Fly Ash		Slag		Fly Ash			Slag
			15.0	37.4	25.2	50.2	30.1			50.0
A	Saturated Cover	NaCl	**32.5**	**35.0**	25.0	42.5	**42.5**	–	–	42.5
B		NaCl and $MgCl_2$	**30**	**35.0**	25.0	35.0	–	**32.5**	–	42.5
C		Residual NaCl from Spreader and Traffic Spray	**35.0**	**35.0**	30.0	45.0	–	–	**35.0** **42.5**	42.5
D	ASTM C1315 (Curing and Sealing Compound)	NaCl	**35.0**	**35.0**	30.0	42.5	–	**35.0**	–	42.5
E		NaCl and $MgCl_2$	**32.5**	**35.0**	25.0	35.0	–	–	**32.5**	42.5
F		Residual NaCl from Spreader and Traffic Spray	**30.0**	**35.0**	30.0	40.0 40.0	–	–	**35.0**	42.5
G	No Curing	NaCl	**35.0**	**35.0**	25.0	42.5	–	**32.5**	–	42.5
H		NaCl and $MgCl_2$	**30.0**	**35.0**	25.0	35.0	–	–	30.0	40.0
I		Residual NaCl from Spreader and Traffic Spray	**30.0**	**40.0** **35.0**	25.0 32.5	40.0	–	–	**35.0** **40.0**	40.0 40.0

Notes:
[1] Cells in red indicate values that exceed selected criteria.
[2] Cement concrete sidewalk panels containing Mix Design Formulation No. 4 were subjected to prohibited placing and finishing practices to study its effect on scaling.

Table 6.4: Water-cementitious ratio results from petrographic analysis

Panel Group	Curing Method	Deicing Method	ACI 201.2R w/cm Criteria: ≤ 0.45							
			Mix Design Formulation No.							
			1	**2B**	**3**	**4[1]**	**5**	**5B**	**5C**	**6**
			0.43	**0.49**	**0.43**	**0.42**	**0.42**	**0.44**	**0.45**	**0.44**
A	Saturated Cover	NaCl	0.44	**0.48**	0.44	0.43	0.43	–	–	0.44
B		NaCl and $MgCl_2$	0.45	**0.48**	0.44	0.43	–	0.44	–	0.44
C		Residual NaCl from Spreader and Traffic Spray	0.44	**0.48**	0.44	0.42	–	–	0.44	0.45
									0.44	
D	ASTM C1315 (Curing and Sealing Compound)	NaCl	0.45	**0.47**	0.43	0.43	–	0.43	–	0.44
E		NaCl and $MgCl_2$	0.44	**0.50**	0.43	0.43	–	–	0.44	0.44
F		Residual NaCl from Spreader and Traffic Spray	0.44	**0.48**	0.44	0.43	–	–	0.45	0.44
						0.43				
G	No Curing	NaCl	0.44	**0.49**	0.44	0.42	–	0.45	–	0.43
H		NaCl and $MgCl_2$	0.45	**0.50**	0.43	0.43	–	–	**0.46**	0.45
I		Residual NaCl from Spreader and Traffic Spray	0.44	0.45	0.43	0.43	–	–	0.44	0.43
				0.49	0.44				0.45	0.43

Notes:
[1] Cement concrete sidewalk panels containing Mix Design Formulation No. 4 were subjected to prohibited placing and finishing practices to study its effect on scaling.

Table 6.5: Paste content results from petrographic analysis

Panel Group	Curing Method	Deicing Method	AASHTO PP 84 Paste Content (%) Criteria: ≤ 28.0							
			Mix Design Formulation No.							
			1	2B	3	4[1]	5	5B	5C	6
			29.7	27.0	29.6	29.7	27.3	28.1	28.2	27.9
A	Saturated Cover	NaCl	30.2	27.6	29.7	34.0	32.1	–	–	31.5
B		NaCl and $MgCl_2$	36.9	30.2	34.8	35.5	–	30.2	–	33.0
C		Residual NaCl from Spreader and Traffic Spray	35.0	30.3	33.8	34.0	–	–	32.8	32.3
									31.4	
D	ASTM C1315 (Curing and Sealing Compound)	NaCl	32.5	28.1	32.8	32.0	–	29.1	–	32.1
E		NaCl and $MgCl_2$	37.4	29.9	33.1	33.7	–	–	31.3	29.0
F		Residual NaCl from Spreader and Traffic Spray	39.5	30.2	37.3	30.0	–	–	32.5	34.4
						35.5				
G	No Curing	NaCl	33.3	31.1	33.9	35.0	–	36.2	–	27.5
H		NaCl and $MgCl_2$	37.2	29.8	35.9	35.3	–	–	30.0	30.4
I		Residual NaCl from Spreader and Traffic Spray	31.1	27.4	31.9	34.8	–	–	30.3	35.5
				27.5	32.6				33.0	33.1

Notes:
[1] Cement concrete sidewalk panels containing Mix Design Formulation No. 4 were subjected to prohibited placing and finishing practices to study its effect on scaling.

Table 6.6: Air content results from petrographic analysis

Panel Group	Curing Method	Deicing Method	ACI 201.2R Air Content (%) Criteria: 7.0 ± 1.5							
			Mix Design Formulation No.							
			1	2B	3	4[1]	5	5B	5C	6
A	Saturated Cover	NaCl	6.1	7.3	8.2	7.0	6.3	–	–	**9.0**
B		NaCl and $MgCl_2$	6.8	6.6	**4.5**	7.0	–	6.5	–	5.7
C		Residual NaCl from Spreader and Traffic Spray	6.5	6.2	6.9	5.8	–	–	7.6	**4.9**
									6.0	
D	ASTM C1315 (Curing and Sealing Compound)	NaCl	5.7	6.8	6.0	7.8	–	7.3	–	7.2
E		NaCl and $MgCl_2$	5.7	**9.0**	**4.5**	5.3	–	–	6.6	5.5
F		Residual NaCl from Spreader and Traffic Spray	5.9	**5.3**	5.9	6.2	–	–	6.8	5.7
						7.3				
G	No Curing	NaCl	7.0	7.7	7.0	5.7	–	6.7	–	6.5
H		NaCl and $MgCl_2$	5.7	6.5	6.8	**5.3**	–	–	7.2	6.6
I		Residual NaCl from Spreader and Traffic Spray	5.5	7.0	5.4	7.2	–	–	5.8	6.5
				6.5	7.0				6.1	7.3

Notes:
[1] Cement concrete sidewalk panels containing Mix Design Formulation No. 4 were subjected to prohibited placing and finishing practices to study its effect on scaling.

Table 6.7: Air void system spacing factor results from petrographic analysis

<table>
<tr><th rowspan="4">Panel Group</th><th rowspan="4">Curing Method</th><th rowspan="4">Deicing Method</th><th colspan="8">Air Void System Spacing Factor (in.)</th></tr>
<tr><th colspan="8">Mix Design Formulation No.</th></tr>
<tr><th>1</th><th>2B</th><th>3</th><th>4[1]</th><th>5</th><th>5B</th><th>5C</th><th>6</th></tr>
<tr><th colspan="8">ACI 201.2R Spacing Factor Criteria: ≤ 0.008</th></tr>
<tr><td>A</td><td rowspan="4">Saturated Cover</td><td>NaCl</td><td>0.006</td><td>0.006</td><td>0.005</td><td>0.005</td><td>0.007</td><td>–</td><td>–</td><td>0.006</td></tr>
<tr><td>B</td><td>NaCl and $MgCl_2$</td><td>0.007</td><td>0.007</td><td>0.005</td><td>0.006</td><td>–</td><td>0.007</td><td>–</td><td>0.006</td></tr>
<tr><td rowspan="2">C</td><td rowspan="2">Residual NaCl from Spreader and Traffic Spray</td><td rowspan="2">0.007</td><td rowspan="2">0.007</td><td rowspan="2">0.006</td><td rowspan="2">0.006</td><td rowspan="2">–</td><td rowspan="2">–</td><td>0.007</td><td rowspan="2">0.007</td></tr>
<tr><td>0.007</td></tr>
<tr><td>D</td><td rowspan="4">ASTM C1315 (Curing and Sealing Compound)</td><td>NaCl</td><td>0.006</td><td>0.006</td><td>0.005</td><td>0.005</td><td>–</td><td>0.007</td><td>–</td><td>0.006</td></tr>
<tr><td>E</td><td>NaCl and $MgCl_2$</td><td>0.007</td><td>0.008</td><td>0.005</td><td>0.005</td><td>–</td><td>–</td><td>0.007</td><td>0.007</td></tr>
<tr><td rowspan="2">F</td><td rowspan="2">Residual NaCl from Spreader and Traffic Spray</td><td rowspan="2">0.007</td><td rowspan="2">0.007</td><td rowspan="2">0.006</td><td>0.005</td><td rowspan="2">–</td><td rowspan="2">–</td><td rowspan="2">0.007</td><td rowspan="2">0.007</td></tr>
<tr><td>0.007</td></tr>
<tr><td>G</td><td rowspan="4">No Curing</td><td>NaCl</td><td>0.007</td><td>0.008</td><td>0.006</td><td>0.005</td><td>–</td><td>0.007</td><td>–</td><td>0.006</td></tr>
<tr><td>H</td><td>NaCl and $MgCl_2$</td><td>0.007</td><td>0.007</td><td>0.006</td><td>0.006</td><td>–</td><td>–</td><td>0.007</td><td>0.007</td></tr>
<tr><td rowspan="2">I</td><td rowspan="2">Residual NaCl from Spreader and Traffic Spray</td><td rowspan="2">0.006</td><td>0.007</td><td>0.006</td><td rowspan="2">0.005</td><td rowspan="2">–</td><td rowspan="2">–</td><td>0.006</td><td>0.008</td></tr>
<tr><td>0.009</td><td>0.006</td><td>0.008</td><td>0.007</td></tr>
</table>

Notes:
[1] Cement concrete sidewalk panels containing Mix Design Formulation No. 4 were subjected to prohibited placing and finishing practices to study its effect on scaling.

Table 6.8: Air void system specific surface area results from petrographic analysis

Panel Group	Curing Method	Deicing Method	Air Void System Specific Surface Area (in^2/in^3)							
			Mix Design Formulation No.							
			1	2B	3	4[1]	5	5B	5C	6
			ACI 201.2R Specific Surface Area Criteria: ≥ 600							
A	Saturated Cover	NaCl	720	632	686	932	631	–	–	632
B		NaCl and $MgCl_2$	646	612	1083	796	–	632	–	797
C		Residual NaCl from Spreader and Traffic Spray	660	631	795	900	–	–	**585** 697	752
D	ASTM C1315 (Curing and Sealing Compound)	NaCl	782	728	930	817	–	**536**	–	746
E		NaCl and $MgCl_2$	730	**430**	1083	991	–	–	624	700
F		Residual NaCl from Spreader and Traffic Spray	727	688	863	894 650	–	–	699	703
G	No Curing	NaCl	615	**538**	797	993		680		719
H		NaCl and $MgCl_2$	730	**595**	842	958	–	–	627	666
I		Residual NaCl from Spreader and Traffic Spray	777	**593** **486**	825 720	865	–	–	790 **576**	**572** 654

Notes:
[1] Cement concrete sidewalk panels containing Mix Design Formulation No. 4 were subjected to prohibited placing and finishing practices to study its effect on scaling.

The following commentary are direct excerpts from the *WJE Petrographic Studies of Concrete Cores* (see Appendix I for complete report).

The WJE report states that:

> As described in greater detail in the datasheets, the concrete represented by the cores was generally well consolidated, with scattered entrapped air voids frequently observed in many cores. Distribution of the material constituents was generally uniform in the body of the concrete below a weak top surface layer, which is described separately below. All cores consisted of the same or similar crushed siliceous rock coarse and fine aggregate dispersed in a generally well air-entrained cementitious paste of either Portland cement and fly ash or Portland cement and slag cement. No evidence of distress was observed in the body of the

concrete. No evidence of deleterious expansive reactions involving the aggregates, such as alkali-silica reaction (ASR), was observed.

The information provided in Sections 6.2.1 through 6.2.7 applies to all cores unless stated otherwise, as indicated in the WJE report.

6.2.1 Aggregate

The coarse aggregate in all 60 cores is composed of crushed siliceous metamorphic and igneous rocks, including quartzite, granofels, schist, granite, quartz diorite/tonalite, and gneiss. Many of the coarse aggregate particles contain minor to significant amounts of pyrite and mica. The coarse aggregate particles are angular to subangular, and mainly blocky to occasionally elongate/flat or near-elongate/flat. Elongate/flat particles generally did not show preferred orientation, except in a few cores (e.g. Core 37, Core 56). Coarse aggregate particles are pale-white to dark-gray, greenish-gray, and brown in color. The observed nominal top size of the aggregate is 1/2, 3/4, or 1 inch (as compared to the 3/4-inch maximum aggregate size specified in all six mix designs). Cores with observed 1/2-inch nominal top size aggregates generally had a higher paste volume but a lower coarse aggregate volume.

Fine aggregate is composed of sand-sized particles likely manufactured from siliceous rocks of the same or similar lithology as the coarse aggregate. The sand particles resemble the coarse aggregate in color, composition, and texture, with increasing amounts of rock-forming minerals including mainly quartz, feldspar, and mica. White mica (muscovite) flakes are commonly observed; the elongated mica flakes frequently aligned along a specific direction, likely related to placing and vibrating.

Small portions of quartz particles and quartz grains in the siliceous rocks appeared to be physically strained and are potentially reactive with alkalis from Portland cement (alkali-silica reaction (ASR)). However, both coarse and fine aggregate particles are generally in good condition, free of excessive micro-cracks. A few cores contained fine aggregate particles that were naturally internally fractured in the near-surface region of the samples (e.g., Core 20 and Core 26). No evidence of ASR was observed (not expected for the new concrete). The presence of supplementary cementitious materials (SCMs) is expected to assist with mitigating potential ASR. Certain forms of pyrite, observed primarily in the coarse aggregate particles, may cause staining and discoloration when it occurs in near-surface particles, which is considered to be an aesthetic issue.

Laboratory-induced freshly fractured surfaces passed both around and through the aggregate particles in varying proportions among the cores, indicating an overall moderately strong to strong paste-aggregate bond. Fresh fractured surfaces in the slag-containing concrete appeared to extend through more aggregate particles than the fly ash-containing cores overall, indicating a stronger paste-aggregate bond in the concrete containing slag. The observation appeared to be consistent with a higher w/c for some fly ash mixtures and a low reactivity of the fly ash, compared to the slag cement.

6.2.2 Paste

Cores with slag cement generally exhibited a blue-green color of varying tint/strength that is typical of slag-containing concrete, consistent with different amounts of slag cement in the mixtures (see Figure A.3, Figure A.4, and Figure A.6, WJE report). The blue-green color faded to varying degrees with increasing time of exposure to air. Cores containing fly ash generally exhibit a medium gray color in the body concrete below the weak, light-gray surface layer. No significant difference in color was noted between cores with different dosages of fly ash (see Figure A.1, Figure A.2, and Figure A.5, WJE report).

The paste was generally moderately hard for most of the cores but was moderately soft to moderately hard for a few of the fly-ash containing cores. The paste could generally be scratched using a copper pick (3.0 Mohs scale hardness) with varying amounts of effort. Minor amounts of copper debris were frequently left on the scratched surfaces, suggesting the paste has a hardness close to 3 on the Mohs hardness scale. Hydration of Portland cement appeared to be advanced in general and near complete in a few of the fly ash containing cores (predominantly Mix Design 2 cores), probably due to a higher effective water-to-Portland cement ratio in these cores resulting from the higher dosages of fly ash. Calcium hydroxide crystals were generally small, and the content of calcium hydroxide was generally low, consistent with presence of significant amounts of SCMs. Calcium hydroxide will react with fly ash or slag cement via pozzolanic reactions to form additional calcium-silicate-hydrate (CSH) paste.

6.2.3 Air Void System

Parameters most relevant to predicting the effectiveness of the air-void system to resist distress due to cyclic freezing and thawing include the spacing factor, specific surface area (bubble size), and total air content. The spacing factor is an index related to the maximum distance of any point in the cement paste to the periphery of an air void, expressed in inches. The specific surface area describes the average surface area of air voids, expressed as area in in^2 per unit in^3 of air-void volume; the greater the specific surface, the smaller the air voids, on average. For improved durability in freezing and thawing environments, *ACI 201.2R-16*, *Guide to Durable Concrete,* recommends that hardened concrete have a spacing factor equal or less than 0.008 inch, a specific surface area greater than 600 in^2/in^3, and an air content in the range 5.5% to 8.5% (for concrete with 3/4-inch aggregates exposed to freezing and thawing and deicing salts in service) *(5).*

Air-void system analyses of the 60 cores (and the six cylinders received right after sidewalks were placed) indicate that all cores meet the ACI recommendations for spacing factor and air content, with a few cores (7 out of 60 cores) marginally low in specific surface (meaning the air voids are slightly larger on average than those recommended by *ACI 201.2R-16 (5)*). It is worthwhile to note that the specific surface is a non-additive average parameter and can be biased lower than the recommended 600 in^2/in^3 if the concrete contains large entrapped air voids, even if the small air voids alone in the concrete are sufficient to meet the requirements. The air-void system in all cores is considered to meet the ACI recommendations for freeze-thaw durability. The distribution of entrained air voids is uniform in the body concrete of all cores based on the petrographic examination.

6.2.4 Cracking

No large-scale cracks were observed in any of the cores. Surface-perpendicular and surface-parallel micro-cracks were observed in most of the cores within the top 1/4 to 1/2 inch. The micro-cracking perpendicular to the surface appeared to be related to drying shrinkage, and the micro-cracking parallel to the surface appeared to be likely related to cyclic freeze-thaw. Very short micro-cracks, visible only during thin-section studies were also frequently observed in the concrete body in some cores, appearing more frequently in slag-containing cores than in fly-ash containing cores. The fine micro-cracks may have been related to drying shrinkage or autogenous self-desiccation.

6.2.5 Carbonation

The depth of carbonation varied slightly among the cores but was generally within the top 0.5 to 4.0 mm (0.02 to 0.16 inch) of the surface. The depth of carbonation was not considered to be excessive for a broom-finished surface.

6.2.6 Secondary Deposits

Trace to minor amounts of ettringite, calcium hydroxide, or both were observed lining air voids in the majority of cores. No secondary deposits were observed in a minor proportion of the cores. Air voids were generally clean and free of secondary deposits in these cores. Details can be found in the datasheets contained in the WJE report.

6.2.7 Surface Distress

Near-surface conditions were present that would make concrete near the surface of the cores susceptible to premature freeze-thaw distress and general weathering/erosion. Almost all of the 60 cores exhibited a thin, weak, absorptive layer near the top surface of the concrete. The thickness of the weak porous surface mortar ranged from 0.02 to 0.1 inch. The physical properties (lighter gray color, soft to moderately soft paste, dusting, high absorption, suspected bleed channels, early-age drying shrinkage cracks, and carbonation) and the microscopical characteristics of the paste in the weak surface layer appeared to be generally consistent with elevated w/cm and possibly inadequate or less-than-optimal curing. The estimated w/cm in this near-surface layer typically ranged between 0.50 and 0.55, with a higher range 0.55 to 0.60 more typical in cores representing Mix 2 (which had a higher design w/cm of 0.49). Photographs showing the weak layer and descriptions of the weak top layer can be found in datasheets of the WJE report in Appendix I.

6.2.8 Cement Concrete Mix Design Formulation Comparison

Petrographic studies of the cores revealed that concretes represented by the cores were generally well consolidated with varying amounts of scattered entrapped and consolidation voids observed in the cores. Distribution of aggregate, paste, and air-voids was generally uniform in the body of the concrete below a weak top layer described separately above. The concrete appeared to be in general compliance with the provided respective mix designs, except that the amounts of fly ash appeared to be higher than the specified replacement rates based on thin-section examinations. The estimated water-to-cementitious materials ratios (w/cm) in all cores were close to or less than (Mix 2 cores) specified, suggesting there was probably no significant over-watering or water additions in the field. Measured paste volumes for the cores were generally greater than the paste volumes calculated based on the mixture proportions, while the measured total volumes of aggregate were generally lower (with a few exceptions) than the calculated total aggregate volumes based on the mixture proportions. The calculated volume proportions are affected by the specific gravities used in the calculations. However, the coupled consistent volumetric discrepancies were probably also related to low numbers of large-sized coarse aggregate particles (possibly smaller top size than specified). Brief summaries for the six mix design groups are provided in the following sections.

6.2.8.1 *Mix Design Formulation No. 1*

The measured total aggregate volumes in the group were frequently lower than the specified aggregate volumes (Table C.1, WJE report). The measured paste volumes were generally higher than the specified paste volumes, up to 7.3% higher as calculated in Core 19. The measured hardened air contents were quite close to the plastic air content estimated from the mix design (6.9%), with no more than 1.4% difference. Estimated fly ash contents varied but were within a broad range of 20% to 40% (Table B.1 and Table B.2, WJE report), which was generally higher than specified in the mix design (15%). Causes for the overestimation of fly ash may include:

- Low reactivity of the fly ash that leads to more abundant residual fly ash particles and biases to a visually higher fly ash dosage; abundant black fly ash particles were observed that appeared to be consistent with a low fly ash reactivity.
- Limited, project-specific reference samples appropriate for estimating fly ash content in this mixture.
- Non-uniform distribution of cementitious materials on a microscale and limitations of microscopy arising from the thickness of the thin sections.
- Possibly greater amounts of fly ash batched in the mixtures.

6.2.8.2 *Mix Design Formulation No. 2*

The cores representing Mix 2 had the highest estimated w/cm (0.47 to 0.52; Cores 26 and 32) among the cores, as well as the highest design w/cm (0.49) among the six mix designs. Estimated fly ash content was typically 30% to 40% of total cementitious materials, which is consistent with the 37.4% per the mix design (Table A.2, Table B.3 and Table B.4, WJE report). The measured total aggregate volumes in the group were frequently lower than the specified aggregate volumes (Table C.2, WJE report). The measured paste volumes were generally higher than the specified paste volumes, up to 4.4% higher as calculated in Core 14. The measured hardened air contents were generally lower than the plastic air content estimated from the mix design (10.4%), with a maximum difference of 5.1% (Core 45 had a measured air volume of 5.3%).

6.2.8.3 *Mix Design Formulation No. 3*

The measured total aggregate volume in the group of cores representing Mix 3 were generally lower than specified, by up to 6.8% lower in Core 46 (Table C.3, WJE report). The measured paste volumes were generally higher than the specified, up to 8% higher in Core 46. The measured hardened air contents were generally lower than designed, up to 2.6% lower than the plastic air content estimated from the mix design (7.1%) in Cores 21 and 27. Estimated slag cement contents varied but were within a broad range of 20% to 40% as compared to 25% of the total cementitious materials by mass per the mix design (Table B.5 and B.6, WJE report). The estimated w/cm in the cores varied from 0.40 to 0.46, which was close to specified (0.43). The concrete represented by each core in the group is considered to be consistent with the mix design.

6.2.8.4 *Mix Design Formulation No. 4*

The measured total aggregate volume in the group were frequently less than specified, by up to 5.5% lower in Core 48 (Table C.4, WJE report). Core 47 contained greater amounts of coarse aggregate. The measured paste volumes were generally higher than specified, up to 6.2 % higher in Cores 22 and 48. The measured hardened air contents were generally lower than the plastic air content estimated from the mix design, up to 2.7% lower in Cores 28 and 34. Estimated slag cement contents varied among the cores but were within a broad range of 30% to 50% (Table B.7 and Table B.8, WJE report). The range of w/cm estimates was 0.39 to 0.45 for all cores and was close to specified w/cm (0.42). The concrete represented by each core in the group is considered to be consistent with the mix design.

6.2.8.5 *Mix Design Formulation No. 5*

The measured total aggregate volume in the group were generally lower than specified, while the measured paste volumes were generally higher than the specified, up to 8.6% higher in Core 17 (Table C.5, WJE report). The measured hardened air contents were no greater than 1.1% different from the plastic air content estimated from the mix design (6.7%). Estimated fly ash contents varied within a broad range of 30% to 50% of cementitious materials (Table B.9 and Table B.10, WJE report). The specified fly ash amounts in the mix design was 30%. Causes for the potential overestimation of fly ash are similar to what were discussed above for Mix 1 cores. The broad range of w/cm estimates was 0.40 to 0.48, as compared to 0.42 or 0.46 per the mix design (reportedly water was added onsite).

6.2.8.6 *Mix Design Formulation No. 6*

The measured total aggregate volumes in the cores representing Mix 6 were lower than specified, by up to 7% lower in Core 59 (Table C.6, WJE report). The measured paste volumes were generally higher than the specified, up to 8% higher in Core 59. The measured hardened air contents were generally lower than the plastic air content estimated from the mix design (7.5%), up to 2.7% lower in Core 43. The broad range of estimated slag cement contents were 30% to 50% (Table B.11 and Table B.12, WJE report), as compared to 50% per the mix design. The estimated w/cm from the Mix 6 cores ranged from 0.40 to 0.47, which bracketed the specified w/cm (0.44). The concrete represented by each core in the group is likely consistent with the mix design.

6.2.9 Chloride Ion Concentration

Water-soluble chloride ion concentrations were determined at two depths for each of the concrete core samples. All cores were full-depth cores, measuring between 4-5/8 and 10-1/2 inches in length. Chloride ion concentrations were measured for two 1/4-inch thick slices saw cut from each core: one slice sampled between 3/4 and 1 inch from the top surface, and the other slice sampled between 4-1/2 and 4-3/4 inches from the top surface. Each slice was oven-dried and crushed into a fine powder, then analyzed for water-soluble chloride content in general accordance with ASTM C1218, *Standard Test Method for Water-Soluble Chloride in Mortar and Concrete*.

The measured and calculated chloride ion concentrations are presented in **Error! Reference source not found.** through **Error! Reference source not found.** (WJE report, Appendix I). Measured chloride ion concentrations at the top slice (between 3/4 and 1 inch of the top surface) ranged from 0.053 to less than 0.003% by weight of concrete (0.48 to less than 0.02% by weight of cement). Chloride ion concentrations in the bottom slice (between 4-1/2 to 4-3/4 inches of the top surface) were typically less than 0.005% by weight of concrete (0.06% by weight of cement), but were locally as high as 0.100% by weight of concrete (1.26% by weight of cement) where the bottom slice was located near the base of the core (i.e., closer to chloride-contaminated soil). The relatively low chloride concentrations measured near the top of the cores (between 3/4 and 1 inch) are consistent with a limited chloride exposure over a single winter and would be expected to rise over time with additional applications of deicing salts during winter months.

UMass commentary: The highest chloride concentrations found in the top 3/4 and 1 inch were found in panels in Group I, followed by those in Group F. These two groups correspond to panels that were not cured and cured using a sealer and curing compound, respectively. Furthermore, their location in the test site corresponds to panels that were not intentionally subjected to deicing chemicals but that were periodically splashed with chloride contaminated ice and snow during winter operations (raised sidewalk). Panels in Group C (cured using saturated covers), also located in this same area within the test site did not contain high chloride contents near their surface. It should be noted that analyses were done using extracted cores, so these only represent local conditions within each sidewalk panel.

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7.0 Photographic Examination

Photographic documentation of the sidewalk panels began on November 21, 2019. Individual photos were taken of each panel to maintain a visual record of the panels as conditions changed during the late fall/winter season. The dates of photographic documentation are listed in Table 7.1.

Table 7.1: Dates of panel documentation

Dates of Photography	Equipment Used
November 21, 2019	Samsung Galaxy S10e camera on tripod[1]
November 30, 2019	
January 7, 2020	Canon EOS Rebel T6 DSLR Custom made rig/canopy/lighting
January 24, 2020	
February 21, 2020	
June 7, 2020	

Note: These photographs were not used in the digital analysis presented later. Photographs were used for documentation purposes only, not analysis.

7.1 Human Visual Examination

Visual analysis was completed using the June 7, 2020, photographic documentation to get an overview understanding of damage progression during the winter (if any). Scuffing, discoloration, and scaling were all remarked identified. A cursory estimation of percentage of panel scaled was also performed, based on severities of scaling: low–surface scratches; medium–some coarse aggregate showing, or moderate amount of upper surface layer lost; and high scaling–clear and extensive coarse aggregate showing.

It appears that scaling primarily occurred on the panels located on the east side of the test site (Panel Groups C, F, and I). There is very clear scaling across all Group I panels, many of the Group F panels, and a couple of the Group C panels. Scaling damage on the panels located on the north side of the site (at-grade panels) appeared to be primarily caused by vehicular traffic.

The visual examination results and the corresponding panel group identification letter, curing method, deicing method, and mix design formulation number for each cement concrete panel are reported in Table 7.2.

Table 7.2: Human-based visual examination results

Panel Group	Curing Method	Deicing Method	Percentage Scaled (%)									
			Mix Design Formulation No.								Average	
			1	2B	3	4[1]	5	5B	5C	6	Deicing Method	Curing Method
A	Saturated Cover	NaCl	20	20	5	5	0	–	–	0	8	14
B		NaCl and $MgCl_2$	35	20	10	15	–	10	–	10	17	
C		Residual NaCl from Spreader and Traffic Spray	35	20	20	15	–	–	10	10	18	
D	ASTM C1315 (Curing and Sealing Compound)	NaCl	5	5	10	45	–	45	–	50	27	20
E		NaCl and $MgCl_2$	0	5	10	30	–	–	25	10	14	
F		Residual NaCl from Spreader and Traffic Spray	15	10	15	35	–	–	25	15	19	
G	No Curing	NaCl	35	35	30	55	–	65	–	50	45	29
H		NaCl and $MgCl_2$	25	5	35	20	–	–	0	40	21	
I		Residual NaCl from Spreader and Traffic Spray	25	25	20	20	–	–	10	25	21	
Mix Design Formulation Average			22	16	17	27	21			23		

Notes:
[1] Cement concrete sidewalk panels containing Mix Design Formulation No. 4 were subjected to prohibited placing and finishing practices to study its effect on scaling.

7.2 Computer-Based Photogrammetry

Quantitative analysis of the panels was performed using the MATLAB Digital Imaging extension. The panel photos were first edited to correct for skew and rotation. Images were converted from the traditional red, green, blue (RBG) image storage to a grayscale digital storage. Using a multistep comparison of pixel color in each panel image, a threshold color was selected for each panel which corresponded to a scaled pixel color. It was necessary to analyze each panel separately due to the variation in shading. This variation resulted from the different mix designs, curing method, and amount of ambient light during when panels were photographed. This process subdivided each panel image into gridded regions, derived an unscaled panel color based on the gridded regions, and calculated the difference between each individual pixel and the base panel color. The ratio of the sum of pixels with color difference greater than a threshold to the total number of pixels in the image gave the percentage of scaling within the image. Areas of suspected scaling are highlighted in the image (Figure 7.1). After visual inspection of the image to confirm shadows and surface anomalies are not included in the scaled regions, the percentage scaled area of the panel is calculated. Factors such as shadows, watermarks, and dirt, panel discoloration, nonvisible shadows, and nonstandard wear was observed to affect the accuracy of the method. Currently, user input is necessary to confirm that the selected threshold value correctly works to distinguish scaled regions from non-scaled regions.

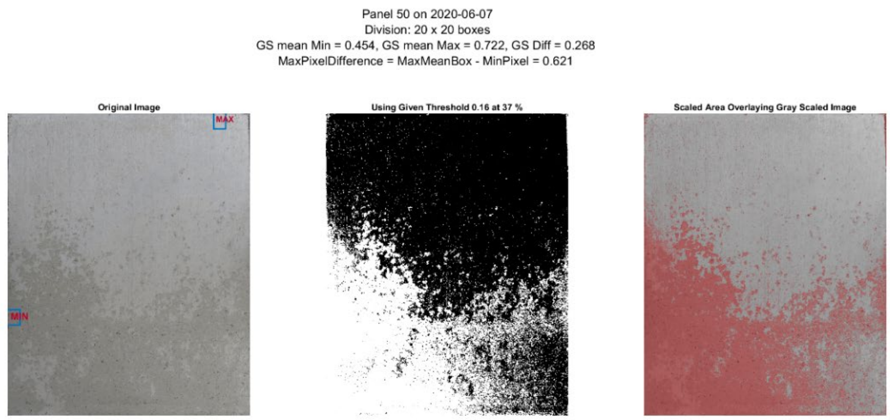

Figure 7.1: Panel 2I (50) after photogrammetric analysis

The computer-based photogrammetry scaling percentage results and the corresponding panel group identification letter, curing method, deicing method, and mix design formulation number for each cement concrete panel are reported in Table 7.3.

Table 7.3: Computer-based scaling percentage photogrammetry results

Panel Group	Curing Method	Deicing Method	Percentage Scaled (%)									
			Mix Design Formulation No.								Average	
			1	2B	3	4[1]	5	5B	5C	6	Deicing Method	Curing Method
A	Saturated Cover	NaCl	6.4	4.3	1.5	3.9	2.7	–	–	3.7	3.8	3.2
B		NaCl and $MgCl_2$	3.0	1.8	2.2	0.9	–	1.9	–	1.5	1.9	
C		Residual NaCl from Spreader and Traffic Spray	2.3	5.2	1.7	4.6	–	–	8.3	1.4	3.9	
D	ASTM C1315 (Curing and Sealing Compound)	NaCl	8.8	4.7	2.1	3.0	–	2.5	–	2.3	3.9	3.1
E		NaCl and $MgCl_2$	1.7	1.7	1.4	0.6	–	–	1.3	1.4	1.4	
F		Residual NaCl from Spreader and Traffic Spray	1.2	6.0	1.4	11.4	–	–	1.8	1.6	3.9	
G	No Curing	NaCl	12.9	1.5	1.8	1.6	–	12.8	–	2.1	5.5	9.0
H		NaCl and $MgCl_2$	3.0	12.6	1.9	0.9	–	–	1.9	1.9	3.7	
I		Residual NaCl from Spreader and Traffic Spray	2.0	36.6	3.8	9.8	–	–	21.3	33.6	17.9	
Mix Design Formulation Average			4.6	8.3	2.0	4.1	6.1			5.5		

Notes:
[1] Cement concrete sidewalk panels containing Mix Design Formulation No. 4 were subjected to prohibited placing and finishing practices to study its effect on scaling.

8.0 Analysis of Results

8.1 Mix Design Formulation

None of the mix design formulations fully met the criteria identified in Table 8.1, which may result in increased susceptibility to freezing, thawing, and deicing damage. Mix Design Formulations No. 1 and No. 3 had the highest conformance (62.5% of desired properties were met) while Mix Design Formulations No. 5, No. 5B, No. 5C, and No. 6 had the lowest conformance (37.5% of desired properties were met).

Table 8.1: Mix design formulation analysis[1]

Property			Mix 1	Mix 2B	Mix 3	Mix 4	Mix 5	Mix 5B	Mix 5C	Mix 6	Criteria
Combined Aggregate Gradation	Tarantula Curve		**No**	**No**	**No**	**No**	**No**	**No**	**No**	**No**	Yes
	Shilstone Workability-Coarseness Chart		Zone II	Zone II	Zone II	Zone II	**Zone III**	**Zone III**	**Zone III**	**Zone III**	Zone II
Paste System	Freezing, Thawing, and Deicing Resistance, w/cm		0.43	**0.49**	0.43	0.42	0.42	0.44	0.45	0.44	≤ 0.45
	SCM Content (%)	Fly Ash	15.0	**37.4**	–	–	**30.1**	**30.1**	**30.1**	–	≤ 25.0
		Slag	–	–	25.2	**50.2**	–	–	–	50.0	≤ 50.0
	Water-Reducing Chemical Admixtures		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
	Paste Content (%)		**29.7**	27.0	**29.6**	**29.7**	27.3	**28.1**	**28.2**	27.9	≤ 28.0[4]
	Paste Content to Void Content Ratio (PC/VC)		**1.1**	**1.0**	**1.1**	**1.1**	**1.0**	**1.0**	**1.0**	**1.0**	1.25 – 1.75[5]
Air Void System	Air Content (%)		See Section 4.2[2]								7.0 ± 1.0
	Air-Entraining Chemical Admixtures		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Conformance (%)[3]			**62.5 (5/8)**	**50.0 (4/8)**	**62.5 (5/8)**	**50.0 (4/8)**	**50.0 (4/8)**	**37.5 (3/8)**	**37.5 (3/8)**	**37.5 (3/8)**	100 (8/8)

Notes:
[1] Cells in red indicate values that exceed selected criteria.
[2] The air content design targets for the mix design formulations were not provided by the cement concrete producer, and therefore, air content design targets were not analyzed for freezing, thawing, and deicing resistance. However, *AASHTO T 152 Air Content of Freshly Mixed Concrete by the Pressure Method* was conducted on each mix design formulation. The analysis of these test results is provided in Section 4.2.
[3] Conformance is determined by dividing the number of passing results by the number of properties being evaluated in the table.
[4] Criteria specified in *AASHTO PP 84-17 Standard Practice for Developing Performance Engineered Concrete Pavement Mixtures (7).*
[5] Criteria recommended by Taylor et al., 2007 *(1).*

8.2 Field Sampling and Testing

None of the mix designs met fully the criteria included in Table 8.2, which may result in increased susceptibility to freezing, thawing, and deicing damage. Mix Design Formulations No. 3 and No. 4 had the highest conformance (75.0% of desired properties met), while Mix Design Formulations No. 2B, No. 5 (including Mixes 5B and 5C), and No. 6 had the lowest conformance (43.8% of selected criteria met).

Table 8.2: Fresh and hardened concrete analysis[1]

Test Method	Property		Mix 1	Mix 2B	Mix 3	Mix 4	Mix 5[2]	Mix 6	Criteria
T 152	Air Content (%)		**5.1**	6.6	5.5	6.1	**5.0[3]**	**5.4**	7.0 ± 1.5
T 309	Concrete Temp. (°F)		63	59	62	70	60	67	50–90
TP 118	Air Void System No.		0.24	0.24	0.23	0.19	0.23	0.25	≤ 0.25
T 22	Compressive Strength (psi)	7 Days	3460	**1460**	3860	2890	**2470**	3350	≥ 2800
		28 Days	4160	**2230**	5030	4850	**3510**	5540	≥ 4000
		56 Days	4960	**2770**	5340	5500	4240	6300	≥ 4000
T 358	Chloride Ion Penetration (kΩ-cm)	28 Days	**9.2**	**9.9**	**17.1**	**18.6**	**10.1**	**20.0**	≥ 21.0
		56 Days	**14.3**	**18.9**	21.4	26.4	**20.1**	29.7	≥ 21.0
T 161 (A)	Rapid Freezing and Thawing Resistance (Durability Factor)		101	91	101	101	101	103	≥ 80
C672[5]	Scaling Resistance via Standard Moist Cure (Rating)		**3.5**	**5.0**	**2.5**	**5.0**	**5.0**	**5.0**	≤ 2.0
C672[5]	Scaling Resistance: Curing using Saturated Cover (Rating)		1.0	**4.5**	2.0	**4.5**	**4.0**	**5.0**	≤ 2.0
C672[5]	Scaling Resistance: Curing using Sealing and Curing Compound (Rating)		**3.0**	**3.5**	2.0	**4.0**	**5.0**	**5.0**	≤ 2.0
C672[5]	Scaling Resistance: No Curing (Rating)[2][7]		1.0	**4.0**	1.0	**5.0**	**3.0**	**5.0**	≤ 2.0
C856	Scaling Resistance: Curing using Sealing and Curing Compound (Rating)[2][6]		**3.0**	**3.5**	2.0	**4.0**	**5.0**	**5.0**	≤ 2.0
	Scaling Resistance: No Curing (Rating)[2][7]		1.0	**4.0**	1.0	**5.0**	**3.0**	**5.0**	≤ 2.0
	SCM Content (%)	Fly Ash	**32.5**	**40.0**	–	–	**37.5**	–	≤ 25.0
		Slag	–	–	30.0	40.0	–	42.5	≤ 50.0
C457	Air Content (%)		**3.6**	8.2	**4.1**	**4.4**	**4.6**	**4.4**	7.0 ± 1.5
	Spacing Factor (in.)		0.006	0.006	0.005	0.005	0.007	0.006	≤ 0.008
	Specific Surface Area (in^2/in^3)		965	**543**	1039	1067	840	868	≥ 600
Conformance (%)[4]			**63.2 (12/19)**	**36.8 (7/19)**	**78.9 (15/19)**	**63.2 (12/19)**	**36.8 (7/19)**	**57.9 (11/19)**	100.0 (16/16)

Note:
[1] Cells in red indicate values that exceed selected criteria.

[2] Hardened concrete testing was only conducted on Mix Design Formulation No. 5. Hardened concrete testing was not conducted on Mix Design Formulations No. 5B and No. 5C. However, due to the multiple instances of water being added on site for those mixes, it is assumed that the hardened concrete test results for those mixes would have revealed test results even more unfavorable than the test results of Mix Design Formulation No. 5.
[3] The air content result for Mix Design Formulation No. 5C was 4.0%, considerably less than the original Mix Design Formulation No. 5 due to the two instances of water being added onsite. Air content testing was not conducted on Mix Design Formulation No. 5B; however, it is assumed based on air content result of Mix Design Formulation No. 5C that Mix Design Formulation No. 5B had an air content ranging between 4.0% and 5.0%.
[4] Conformance was determined by dividing the number of passing results by the number of properties being evaluated in the table.
[5] Withdrawn by ASTM in 2021.

8.2.1 Air Void System

The air content results from *ASTM C457* examination were especially low, apart from Mix Design Formulation No. 2B. However, the examination showed an exceptional spacing factor (*ASTM C457*) and air void system number (*AASHTO TP 118*) which has been shown to correlate well with freezing, thawing, and deicing resistance in the field. In addition to the air test results, *AASHTO T 161* (A) test results showed excellent rapid freezing and thawing durability factors for all six mix design formulations, which validates the excellent air void system results observed.

8.2.2 Reported Batched Material Quantities Versus Material Quantities Estimated from Petrographic Analysis

One key observation that can be made from these test results is the disparity between the material quantities identified on the batch ticket versus the actual material quantities identified from *ASTM C856 Petrographic Examination of Hardened Concrete* (Table 8.3). All of the fly ash mix design formulations exhibited increased fly ash contents between the design content and the actual content, with the most noticeable discrepancy residing with Mix Design Formulation No. 1, where the fly ash content showed an increase of 117% between the design content and the actual content. The slag mix design formulations were within close conformity when compared with the actual content. The paste content for all the mix designs, apart from Mix Design Formulation No. 1, exhibited increased paste contents when examined via ASTM C856. The differences between batch ticket materials quantities and actual materials quantities may be attributed to cement concrete producer mixing issues where disproportionate amounts of aggregate, cement, fly ash, slag, and paste did not mix homogenously prior to or during placement.

Table 8.3: Batch ticket versus petrographic hardened concrete cylinders results

Property		Mix 1	Mix 2B	Mix 3	Mix 4	Mix 5	Mix 6
Fly Ash Content (%)	Design	15.0	37.4	–	–	30.1	–
	Actual	32.5	40.0	–	–	37.5	–
Slag Content (%)	Design	–	–	25.2	50.2	–	50.0
	Actual	–	–	30.0	40.0	–	42.5
Paste Content (%)	Design	29.7	27.0	29.6	29.7	27.3	27.9
	Actual	27.5	28.3	31.5	33.4	33.6	32.2

8.2.3 *ASTM C672 (WITHDRAWN 2021)* Intermediate Scaling Ratings

Under the laboratory moist cure following *ASTM C672 (WITHDRAWN 2021) (8)* (performed by MassDOT RMS Laboratory), Mix Design Formulations No. 2B (40.0% fly ash per *ASTM C856*), No. 4 (40.0% slag per *ASTM C856*), No. 5 (37.5% fly ash per *ASTM C856*), and No. 6 (42.5% slag per *ASTM C856*) exhibited the least resistance to scaling, reaching a scaling rating of 5.0 after only 25 cycles, 15 cycles, 10 cycles, and 5 cycles, respectively. This may be attributed to the higher amounts of supplementary cementitious materials than Mix Design Formulations No. 1 (32.5% fly ash per *ASTM C856*) and No. 3 (30.0% slag per *ASTM C856*). Mix design formulations with higher amounts of supplementary cementitious materials require longer durations of curing to properly mature and combat scaling.

Most specimens cured using saturated covers reached high scaling rating numbers early during the freeze-thaw cycling protocol. Only specimens fabricated using Mixes 1 and 3 remained at or below a rating of 2.0 after 50 freeze-thaw cycles, with Mix 3 specimens reaching 2.0 after 25 cycles and mix 1 specimens reaching 1.0 after 15 cycles. Specimens fabricated using Mixes 2, 4, 5, and 6 reached scaling rating values of 4.0, 4.5, 3.5 and 5.0 after 15 freeze-thaw cycles. The scaling ratings in these groups seemed to stabilize after 25 cycles at rating numbers approximately equal to 4.0, 4.0, 4.0, and 5.0, respectively. Curing specimens using saturated covers seemed to detrimentally affect the scaling performance of those samples with higher fly ash and slag contents, presumably because of their lower hydration and maturity rate. The early application of moist curing and presence of ambient moisture after placement may have created a weak top layer of concrete that contributed to early scaling.

Specimens cured using a curing and sealing compound (compound curing) performed better than specimens using a saturated cover. The best performing mixes were 1 and 3, with specimens from Mix 3 being the only ones satisfying a scaling rating of 2 after 50 freeze-thaw cycles. Specimens from Mix 1 exceeded the 2.0 rating target after 15 cycles and were at 3.0 after 50 cycles indicating a reduction in the degradation rate. Specimens constructed from Mixes 2, 4, 5, and 6 got to scaling ratings of 3.0, 3.5, 4.0, and 4.5, respectively, after 25 freeze-thaw cycles. The same specimens reached ratings of 3.5, 4.0, 5.0, and 5.0, respectively, after 50 freeze thaw cycles. Although the scaling degradation rate decreased with cycling number, the scaling ratings in these specimens continued to increase with cycling perhaps indicating a decreased protection offered by the curing compound. It should be noted that no attempt was made to reapply curing compound during the tests since this was not being done at the test site either. The addition of higher quantities of fly ash and slag seemed to have again contributed to the poorer resistance against scaling in Mixes 2, 4, 5, and 6.

Specimens prepared using no deliberate curing procedure (no cure) exhibited similar trends. The best performing mix design formulations were Mixes 1 and 3, both of which remained at a scaling rating of 2 or below throughout the first 50 freeze-thaw cycles. The drop in rating from 2 to 1.5 or 1.0 from cycle 10 to 15 is attributed to variability in the individual recording the rating. The method has inherent variability since ratings are conducted visually. Mixes 2, 4, 5, and 6 got to scaling ratings of 4.0, 4.5, 3.0, and 5.0, respectively, after 10 freeze-thaw cycles. Again, the addition of higher quantities of fly ash and slag seem to have contributed to the poorer resistance against scaling in these tests.

8.3 Petrographic Analysis

Wiss, Janey, Elstner Associates, Inc. (WJE) performed petrographic analysis on extracted cores from the cement concrete sidewalk panel site. The following paragraphs are direct excerpts from WJE's *Petrographic Studies of Concrete Cores* report (Appendix J contains the complete report).

> Petrographic, air-void system, and chloride ion examinations and analyses were conducted on 60 concrete cores taken from sidewalks slabs to assess the mixes and the cause of premature surface deterioration that has manifested as mainly pitting, various degrees of scaling, and mortar flaking. The surface deterioration appears to be consistent with salt scaling, a progressive form of concrete deterioration that can occur when moisture- saturated concrete surfaces are exposed to freezing and thawing cycles. Scaling due to freezing and thawing can be accelerated by exposure to relatively low concentrations of deicers (e.g., brines containing 2% to 4% deicer by mass of solution) [Kosmatka and Wilson 2016 (12)], and distress due to scaling can initiate as early as the first winter that the concrete is exposed to freezing and thawing and deicing salts. Poor drainage, including ponding of water or contact with saturated soil, can increase the risk of deterioration by scaling.
>
> Adequate air entrainment, low water-to-cementitious materials ratios (w/cm $\leq$ 0.45), and proper curing (at least seven days moist curing) and finishing of the concrete surfaces are known to increase resistance of concrete to scaling distress. Concrete mixtures with high fly ash (>25%) and slag (>50%) contents may be more susceptible to this form of deterioration, as they usually require extended curing durations to achieve a durable surface and may be more susceptible to near-surface carbonation, which has also been linked to an increased risk of salt scaling. Mix Design No. 2, 3, 4, 5, 6 all have SCM contents at or exceeding these limits and, therefore, may be more susceptible to deterioration by deicer scaling.

The depth of the surface defects (pitting, scaling, and mortar flaking) observed in the 60 cores was consistent with deterioration by deicer scaling, and essentially correlates with a thin, weak, carbonated layer at the top region of the concrete. The thickness of the weak surface mortar ranged from 0.5 mm (0.02 inch) to generally less than 2.5 mm (0.1 inch). The physical properties and microscopical characteristics of the paste in the weak surface layer were consistent with a locally elevated w/cm, likely coupled with inadequate or less-than-optimal curing, and contained a lower air content and a visually poor air void system that are considered to negatively affect freeze-thaw durability. Therefore, while the bulk concrete had characteristics consistent with freeze-thaw durable concrete (e.g., moderate w/cm, adequate air entrainment), the weak near-surface region lacks these characteristics and is therefore more likely to exhibit deterioration due to freezing and thawing and deicer scaling. Moreover, an increased porosity due to near-surface carbonation of concrete mixtures containing large replacement levels of fly ash (>25%) and slag (>50%) has been correlated by others *(5)* with an increased risk of salt scaling. The physical properties and microscopical characteristics of the paste in the weak surface layer were moreover consistent with elevated w/cm, likely coupled with inadequate or less-than-optimal curing, which further increase the risk of salt scaling. Additionally, the surface layer also contained a lower air content and a visually poor air void system that are considered to negatively affect freeze-thaw durability.

The most effective methods to reduce the formation of the weak near-surface layer are to ensure that the concrete is (1) properly finished and (2) adequately cured. Concrete that contains cementitious materials such as slag cement or fly ash requires particular attention to finish timing to avoid premature finishing and subsequent water addition to the surface by bleeding. It also requires particular caution towards curing to promote conditions needed for the SCMs to hydrate and initiate pozzolanic reactions. The time of set and the rate of strength gain are typically delayed in concrete that contains slag cement in excess of 25% replacement and may be delayed at even lower dosages for fly ash. Therefore, extended curing durations may be necessary for concrete that contains large volumes of SCMs to ensure adequate strength development prior to exposure to freezing conditions and deicing chemicals. The observed pitting and mortar flaking over the aggregate particles was caused by the elevated w/cm in the weak layer, or by inadequate hydration of the cement and slag in the paste, or both. Effective moist curing normally prevents mortar flaking by supplying moisture needed for hydration. Traditional moist curing, such as with wet burlap, misting, use of sprinkler hoses, or plastic sheeting, is recommended for concrete that exhibits minimal bleeding, rather than use of curing compounds.

The cores exhibited evidence of the application of a curing compound or a sealer; however, curing appears to have been inadequate. The delayed time of set and reduced rate of bleed from fly ash and slag may affect the timing of application of the curing compound and the effectiveness of the curing compound, once applied.

Curing compounds function by minimizing the loss of internal moisture from the concrete, so that it can then be available for hydration. Curing compounds do not supply moisture and are ineffective if the moisture in the concrete already has been lost through evaporation before the compound is applied. Curing compounds also have limited effectiveness in concrete that produces little bleed water. Bleeding refers to the natural process in which free water in concrete mixtures rises up to the surface region while aggregates (heavier) tend to settle down. Bleeding is beneficial to concrete during curing, since bleed water can compensate for moisture loss from the surface and may avoid premature drying; however, excessive bleed water may also cause excessive surface laitance and weakness. The rate of bleeding and the quantity of bleed water are related to the fineness of the slag and fly ash. Fine slag particles (and also fly ash particles) block pores in the concrete, which tends to slow the rate of bleeding and produce a concrete that bleeds less. Bleeding may not occur at all for some low water w/cm mixtures.

The observed pitting and mortar flaking over the aggregate particles is caused by the elevated w/cm in the weak layer, or by inadequate hydration of the cement and slag in the paste, or both. Effective moist curing normally prevents mortar flaking by supplying moisture needed for hydration. Traditional moist curing is recommended for concrete that exhibits minimal bleeding.

The quality of the paste below the weak surface region is good, and the concrete is expected to perform adequately after the thin weak surface layer has eroded or has been removed. The air voids in the body concrete were mostly small and uniformly distributed, resulting in an air-void system that appears adequate to protect the concrete from damage caused by cyclic freezing and thawing.

No evidence of distress was observed in the body of the concrete, below the weak surface and/or the deteriorated near surface paste. Concretes represented by the cores are judged to be in overall compliance with the respective mix designs, with a possibility of a lower than specified aggregate volume and slowly reactive fly ash. Residual fly ash particles were frequently black in color and the amounts of residual fly ash appeared to be greater than anticipated, based on the mix designs and our experience. Additionally, the slag-containing cores often appeared to exhibit overall better paste qualities than the fly ash containing cores.

No clear correlations between mix designs and susceptibility of surface scaling was found. None of the three cores of the Mix 3 group exhibited significant scaling or other surface distress in general; however, these cores also tended to have negligible chloride ion concentrations near their top surfaces, so the concrete may not have been exposed to as aggressive conditions for scaling as the concrete represented by the other cores. Cores of Mix Group 1 also exhibited less surface distress than the other five groups of cores. Mixes 1 and 3 contained the lowest amounts of SCMs of the six mixture designs and may consequently have been finished and cured in a manner that produced a more durable top surface layer.

Our experience has shown that SCMs improve concrete micro-porosity or durability only when the SCMs are compatible with the cementitious systems, properly proportioned, adequately cured, and react properly via hydraulic/pozzolanic reactions.

It should be noted that panels were not subjected to identical environmental conditions, which compounded with differences in mixture design, curing methods, and deicing agent exposure, may have contributed to the differences observed in surface scaling. Panels in Groups 1, 3, and 5 had about 12 days to mature prior to their first exposure to freezing temperatures (see Section 5). Panels in Groups 2, 4, and 6 had only about 7 days prior to being exposed to freezing temperatures. Furthermore, panels of Groups 1 and 3 contained the lowest SCM contents so maturity and strength development would have occurred faster. Group 5 panels contained 30% fly ash and would require a longer time to mature and would have benefitted from an extended curing period. These observations highlight the importance of maturity development by following proper curing practices to allow concrete to develop the necessary strength before being subjected to freezing temperatures.

8.4 Photographic Examination

Results from the photographic analysis presented in Table 7.3 are summarized in this section, with a focus on the influence of curing method, mix design, and exposure to deicing salts. Table 8.4 lists the computed average percentage scaled of the panels depending on the curing method used after placement. As shown, both intentional curing methods used (saturated cover and curing/sealing compound) influenced the observed percentage scaled equally (3.1% and 3.2%). Panels where no intentional curing was conducted were observed to scale three times more than panels with any of the intentional curing techniques (9.0%). The results in this table represent averages of all the panels, so they were subjected to varied deicing conditions and location in the test site.

Table 8.4: Computer-based photogrammetry results by curing method

Ranking	Curing Method	Average Percentage Scaled (%)
1	Curing and Sealing Compound (ASTM C1315)	3.1
2	Saturated Cover	3.2
3	No Curing	9.0

Table 8.5 presents average measured scaling results by mix design formulation. These results indicate that Mix Design No. 3 was the best performing mix on average with a 2.0% scaling, while Mix Designs No. 4, No. 1, No. 6, and No. 5 ranged from 4.1 to 6.1% scaling. Mix Design No. 2B performed the worst on average with 8.3% scaling. This mix contained 37%

fly ash replacement because of a batching error as discussed previously. The two mixes containing the highest contents of fly ash (Mixes No. 5 and No. 2) presented the highest average scaling values. However, the differences in measured average scaling are small so for the range of SCM replacements used, it does not appear that they had a significant detrimental influence on scaling performance.

Table 8.5: Computer-based photogrammetry results by mix design formulation

Ranking	Mix Design Formulation No.	SCM Content (%)	Average Percentage Scaled (%)
1	3	Slag (25.2)	2.0
2	4[1]	Slag (50.2)	4.1
3	1	Fly Ash (15.0)	4.6
4	6	Slag (50.0)	5.5
5	5	Fly Ash (30.1)	6.1
6	2B	Fly Ash (37.4)	8.3

Notes:
[1] Cement concrete sidewalk panels containing Mix Design Formulation No. 4 were subjected to prohibited placing and finishing practices to study its effect on scaling.

The effects of deicing application on average measured scaling are summarized in Table 8.6. Panels exposed to sodium chloride (NaCl) used to treat the road at the test site in combination with periodic hand application of $MgCl_2$ were observed to scale the least at 2.3%, followed by panels exposed only to NaCl from road treatment with an average measured scaling of 4.4%. Panels that were covered periodically with snow or ice containing NaCl from roadway treatment (residual NaCl and traffic spray) had the highest average measured scaling at 8.6%. There may be two compounding factors that explain these results:

- Panels subjected to residual NaCl and traffic spray were located at the east side of the test site, where grading is primarily flat and where sidewalk panels were cast against a small embankment allowing chloride-laden snow to remain on the panels.
- Panels subjected to NaCl or NaCl/$MgCl_2$ deicing agents were cast flush with the roadway pavement on a sloping area of the test site, allowing water to run off more easily from the panels.

As a result, the panels on the east side were subjected to freeze-thaw cycles under moist conditions with water containing chlorides, primarily in the immediate period after snow events.

Table 8.6: Computer-based photogrammetry results by deicing method

Ranking	Deicing Method	Average Percentage Scaled (%)
1	Sodium Chloride (NaCl) and Magnesium Chloride ($MgCl_2$)	2.3
2	Sodium Chloride (NaCl)	4.4
3	Residual Sodium Chloride (NaCl) from Spreader and Traffic Spray	8.6

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9.0 Recommendations

Through the literature review process, mix design formulation analysis, construction practices before and during concrete placement, experimentation of best finishing practices compared with prohibited finishing practices, varying curing methods, varying deicing agent applications, and petrographic analysis of hardened concrete, the recommendations presented in this chapter are provided with the goal of preventing cement concrete sidewalk surface scaling deterioration.

This research project used *ASTM C672 (WITHDRAWN 2021)* as a means to quantify scaling of specimens prepared under various curing methods including the range of conditions encountered in the field *(8)*. However, the ASTM standard test method was withdrawn in early 2021, after testing was completed. *ASTM C672 (WITHDRAWN 2021)* has been reported as overly aggressive on mix designs containing supplementary cementitious materials. Mixes with supplementary cementitious materials set later and develop strength properties at a lower rate than cement-based mixes, neither of which is addressed in the finishing and curing procedures of *ASTM C672 (WITHDRAWN 2021)*. Therefore, the results from this test method are not to be used as an absolute measure of scaling resistance, as the test method is no longer an ASTM standard and does not correlate well to field performance.

Surface scaling of concrete sidewalks is largely influenced by the properties and strength of the top layer of concrete that extends a few millimeters into the body of the concrete sidewalk (typically 3 to 6 mm [0.12 to 0.2 inch]). This layer needs to have adequate strength, excellent air void structure, low w/cm content to withstand the rigors of freeze-thaw cycles combined with application of deicing materials. To achieve durable performance of sidewalks, materials, construction procedures, and maintenance operations must all be closely controlled. Mix design, quality control during production of concrete, finishing, curing, and protection of concrete particularly at its early ages is paramount for good performance. The large number of organizations involved in this study reflects the collaborations that must exist in production of durable sidewalks in cold weather regions.

To ensure a high quality and durable concrete sidewalk is produced and constructed, both contractor quality control (QC) and department acceptance must be conducted. QC must be established, maintained, and performed by the contractor (and sub-contractors) to monitor, assess, and adjust production and construction processes, maintain continuous control of the process, and ensure that the final material or product will meet the specified level of quality. QC must be incorporated into all stages of cement concrete sidewalk production and construction, including control, handling, and storage of constituent materials, mix design formulation, batching, mixing, transporting, sub-grade preparation, placement, finishing, curing, cold weather concreting, and hot weather concreting. Acceptance must be performed by the department to evaluate the degree of compliance with contract requirements, monitor contractor and sub-contractor QC activities, and determine the acceptability of all materials produced and placed.

9.1 Mix Design Formulation

The mix design formulation should be developed in accordance with Taylor et al., 2007 *(1)*. Specific recommendations on mixture design are presented as follows.

9.1.1 Combined Aggregate System

1. **Concrete mixtures should be based on a combined aggregate system that meets the fine and coarse aggregate size distributions satisfying the Tarantula Curve indicated in Section 3.1.1 and the workability coarseness in Section 3.1.2.** The mix optimization tools provided in Chapter 3 are recommended for use. The use of these tools has been shown to produce concrete mixtures with good workability while optimizing paste and aggregate contents. Results from the petrographic analysis indicated that all cores had a higher calculated paste content and lower calculated aggregate content than what was specified in the concrete mixture design, probably as a result of a smaller maximum aggregate size than aggregates satisfying the Tarantula Curve.

9.1.2 Paste System

1. **The paste volume is recommended to be kept below 28% to control cracking induced by shrinkage, while maintaining adequate workability.** Control of the maximum paste content is an important step in mixture optimization. The recommended limit has been shown to produce mixtures with adequate workability when a combined aggregate system is optimized according to techniques presented in Chapter 3 (*AASHTO PP 84-17*) *(7)*.

9.1.3 Air Void System

1. **The concrete mixture should contain entrained air consistent with the nominal maximum aggregate size (NMAS) in accordance with *ACI 201.2R-16* for concrete exposed to freeze-thaw cycles in the presence of deicing salts (7.0% for this project)**. To achieve durable concrete, *ACI 201.2R-16* recommends entrained air contents based on NMAS and specific exposure conditions anticipated in the field *(5)*. A measured field variation of +/-1.5% is allowed from the target air entrainment value *(5)*. The air content measured in fresh concrete in three of the six mixes was lower than the recommended value of 7.0+/-1.5% for concrete with exposure category F3a, as specified by the *ACI 201.2R* guide (see Table 4.5). The petrographic analysis revealed that the air void system was adequate as indicated below.

2. **The air-void system should have a maximum spacing factor $\bar{L}$ of 0.008 in. and a minimum specific surface α of 600 in^2/in^3 as recommended by *ACI 201.2R-16 (5)*.** The petrographic analysis of 60 extracted cores indicated adequate values of average air bubble spacing factor and surface area factor for durable concrete within the body of the cores analyzed.

9.1.4 Supplementary Cementitious Materials (SCMs)

1. **Concrete mixtures that do not contain reactive aggregates do not need to contain SCMs.** Reactive aggregates are those containing silicates that may trigger alkali-silica reaction (ASR). SCMs are used to mitigate ASR when using reactive aggregates. However, the use of SCMs poses other challenges during construction because of the slower hydration rate of concrete containing these materials.
2. **Supplemental cementitious materials (SCMs) should comply with applicable ASTM and AASHTO standards and be tested for quality control in accordance with those standards.** Fly ash should comply with *ASTM C618 (13)* and slag should comply with *ASTM C989 (14)*. *ACI 201.2R* lists applicable ASTM standards for other SCMs used in the mixture *(5).* The petrographic analysis found the potential for low-reactivity fly ash in the concrete cores analyzed. The sources of fly ash used should be documented to ensure proper reactivity in concrete. Fly ash to be used in concrete should be tested in accordance with *ASTM C311 (15)* for adequate quality control. As indicated in Section 8.3 of this report, "SCMs improve concrete micro-porosity or durability only when the SCMs are compatible with the cementitious systems, properly proportioned, adequately cured, and react properly via hydraulic/pozzolanic reactions."

3. **Proportioning of concrete mixes containing supplementary cementitious materials (fly ash, slag) must be carefully controlled, given the sensitivity of initial setting time and finishing time on percentage of cement replacement.** For concrete under exposure class F3a (concrete subjected to freeze-thaw cycles in the presence of salts), *ACI 201.2R-16* limits fly ash to a maximum of 25% of the total cementitious materials by mass *(5).* For these same exposure conditions, *ACI 201.2R-16* limits slag to a maximum of 50% of the total cementitious materials by mass *(5).* Better scaling resistance was observed in specimens constructed using mix formulations containing the lowest amounts of fly ash and slag in this study (Mixes 1 and 3, respectively; see Sections 4.3 and 8.2.3). Mix 1 contained 15% fly ash replacement, and Mix 3 contained 30% slag replacement. It does not appear that higher fly ash or slag contents than those currently permitted in *ACI 201.2R-16*, 25% and 50%, respectively, are justified for use at this time. Field observations and the photographic analysis revealed that regardless of curing method, specimens constructed using Mixes 1 and 3 consistently exhibited better scaling performance than those constructed using Mixes 2, 4, 5, or 6. Mixes 2, 4, 5, and 6 contained approximately twice as much fly ash or slag (in percent, by total cementitious material mass) compared with Mixes 1 and 3.

9.1.5 Water-Cementitious Ratio and Concrete Strength

1. **Design concrete mixtures with a w/cm ≤ 0.45.** Although a maximum w/cm ratio of 0.45 is recommended in the *ACI 201.2R* guide for concrete exposed to freezing-thawing cycles and deicing chemicals *(5),* the petrographic analysis conducted in this research identified higher w/cm ratios in most of the extracted cores, particularly near the surface. This could be the result of early finishing, improper curing, or a

combination thereof. Therefore, a design w/cm ratio ≤ 0.45 is recommended to help offset the effect that a higher water content may have on concrete strength, particularly in the upper layers of sidewalk panels.

The reduction in the time required for concrete to produce maturity at which capillaries become discontinuous with decreasing w/cm is reported in *ACI 201.2R-16 (5),* based on results from Powers et al. (see *ACI 201.2R,* Table 3.3.2). For w/cm of 0.4, the age to produce maturity of concrete is estimated as three days for continuously moist-cured concrete, whereas for a w/cm of 0.45 concrete requires seven days to reach maturity for the same curing conditions. Lower w/cm will decrease permeability of concrete and increase strength in a shorter period for equal curing conditions and is likely to result in better durability.

2. **A minimum average compressive strength of concrete f'_c = 4500 psi should be reached prior to first exposure to freezing-thawing in accordance with *ACI 201.2R-16.*** The strength limit given in *ACI 201.2R-16* is intended to provide sufficient strength to avoid cracking of concrete if subjected to tensile stresses that result from water expansion *(5).* Given that MassDOT specifies concrete compressive strength in 1000 psi increments, producers should design their concrete mixtures for a minimum f'_c = 5000 psi to achieve the recommended strength specified in *ACI 201.2R-16*.

9.1.6 Chemical Admixtures

1. **Use of air entraining and water-reducing admixtures (Table 3.3) are recommended.** Air entraining and water-reducing admixtures were used successfully to achieve specific desirable concrete properties. An air entraining admixture satisfying *AASHTO M 194* (Type A) was used to achieve a target air content and an air void structure for durable concrete. The recommended air content for concrete with a NMAS of ¾ in. per *ACI 201.2R-16* was 7.0% with a maximum departure of +1.5% *(5).* The water-reducing admixture satisfied *AASHTO M 154* (P-AEA) and was primarily incorporated for adequate workability. The continued use of water reducing admixtures in future projects is recommended to achieve the lower w/cm recommended above. Use of other admixtures should be evaluated and tested in trial mixtures prior to use in future projects.

9.2 Concrete Delivery, Placement, Finishing and Curing

9.2.1 Concrete Batching and Delivery

Concrete delivered at the job site must conform to *AASHTO M-157* and *ASTM C94/C94M-20 – Standard Specification for Ready-Mixed Concrete*. The following recommendations, which are directly based on this standard, are specifically made because of deviations observed during concrete delivery and placement during this project.

1. **Information in the batch ticket should conform to requirements of *AASHTO M-157*.** This information should be clearly presented so that the mixture characteristics can be quickly identified at the site (batching time, time in transit, number of yards batched, constituent quantities, water withheld [if any], etc.). Concrete not conforming to the concrete mixture design should be rejected.

2. **Any water added in transit or at the job site must be done following the procedures stipulated in *AASHTO M-157*.** The maximum quantity of water may not exceed the maximum water content specified in the design concrete mixture proportions. Any withheld water added at the job site should be added before concrete is discharged from the truck and following the mixing requirements in *AASHTO M-157*. A small amount of concrete can be collected prior to adding any withheld water, to conduct slump or slump flow tests to determine the need to add any withheld water.

9.2.2 Concrete Placement and Finishing

Sections 5.3.2 and 5.3.4 of *ACI 301-16 – Specifications for Structural Concrete (16),* in addition to the recommendations in Prenger, 2018 *(17),* must be followed for concrete placement and finishing. We recommend placing concrete a minimum of one month before temperatures are anticipated to fall below 50º F to allow concrete to develop its 28-day nominal strength without the need to protect it against cold temperatures. Data presented in Figure 5.1 indicate that temperatures fell below freezing only after five days of placement, which indicated that concrete protection practices in accordance with *ACI 301-16* should have been followed *(16).*

If low temperatures are anticipated prior to concrete maturing and development of adequate strength, *ACI 306R-16* §7.3 recommends protecting young concrete against low temperatures until the design compressive strength is reached. Concrete protection against moisture loss and low temperatures will promote adequate concrete durability. Cold weather concrete placement and protection practices in accordance with *ACI 306.1* should be followed in these cases to maintain concrete temperatures, within the limits specified in Chapter 5 of *ACI 306.1 (9)*. The following cold weather concrete placement practices are highlighted because they are specifically applicable to the practices observed in the execution of this project.

1. **For sidewalks, a minimum concrete temperature of 55º F should be maintained during the curing period if the average of highest and lowest ambient temperature is expected to be below 40º F for three successive days or more after placement.** This practice is recommended to allow concrete to develop enough strength during the curing period so that its scaling resistance will improve. If the temperature differential between the concrete surface and the surrounding environment is too high, rapid evaporation may occur causing excessive shrinkage and surface drying.

2. **Concrete sidewalks should not be placed if temperatures are expected to fall below freezing within 10 days after placement.** If young concrete is subjected to ambient temperatures below freezing, any free water present near the surface may

generate excessive tension stresses that may trigger popouts or scaling as water expands during freezing. Application of deicing chemicals during this period will exacerbate the problem. *ACI 201.2R-16* indicates that a single freezing event may not be deleterious to newly placed concrete sidewalks, as long as concrete has reached a strength of 500 psi *(16)*.

3. **Finishing of concrete should be following guidance in Section 2.2.3.1 of this report, and other applicable industry references (*ACI 301-16* and *ACI 201.2-16,* in addition to Prenger, 2018).** ACI 301 specifies general requirements for finishing operations, including the requirement for at least one certified finisher per finishing crew. *ACI 201.2-16* § 4.2.4.3 provides guidance for finishing operations that promote concrete durability. Finishing practices play a very important role in scaling performance of sidewalks subjected to aggressive freeze-thaw environments. Finishing operations should not begin before concrete sets up and while bleed water is present on the surface of concrete *(16)*. Identifying the time for finishing operations to begin is crucial. This gets complicated when SCMs are used in the concrete mixture because of the longer times required for the concrete to set. For mixtures containing fly ash, bleed water migrates and dissipates at a slower rate than concrete without other cementitious materials. In those cases, care should be exercised to avoid finishing the surface prematurely and thereby entrapping water in the topmost layer of sidewalks.

4. **Avoid using prohibited tools to finish sidewalks.** This study examined the use of magnesium (allowed) and steel (prohibited) tools to finish concrete sidewalks. The literature recommends avoiding hard trowel operations (typically conducted using steel tools), particularly for air entrained concrete, to avoid creating a tightly sealed concrete surface that entraps bleed water and creates a weak top surface of concrete. Therefore, the research team recommends the use of magnesium trowels.

9.2.3 Concrete Curing

Section 5.3.6 of *ACI 301-16 – Specifications for Structural Concrete* must be followed for concrete curing *(16)*. For cold or hot weather concreting, curing practices should follow *ACI 306.1-90 – Standard Specification for Cold Weather Concreting* or *ACI 305.1-14 – Specification for Hot Weather Concreting*, respectively *(9,18)*. These two specifications point to cold or hot weather concreting practices that are recommended in *ACI 306R-16 – Guide to Cold Weather Concreting* and *ACI 305R-20 – Guide to Hot Weather Concreting*, respectively *(19,20)*. The contractor should follow curing practices in these documents depending on sidewalk placement conditions to ensure proper protection after concrete placement. These guides provide recommendations for use of different curing methods, protection of concrete after placement, control of evaporation (hot weather), control of extreme temperature drop (cold weather), curing duration, and necessary thermal protection for cold weather concreting, among other recommendations. The contractor should study these recommendations in detail and submit the concreting practices that will be followed, including curing method and protective measures of concrete following placement, in a specific project for approval by MassDOT.

There are advantages and disadvantages on the use of saturated covers or liquid membrane-forming curing and sealing compounds to cure concrete. If using saturated covers, the contractor must ensure that adequate moisture and temperature is maintained throughout the curing period. This implies that the contractor must periodically visit the site to ensure the saturated covers have maintained moisture and are properly positioned. Surface staining may result if the cover is not washed or if the cover is positioned too early after concrete placement.

If liquid membrane-forming curing and sealing compounds are used, the product has to be applied strictly following the manufacturer recommendations. Furthermore, the product should not be applied prior to full elimination of bleed water from the concrete surface. This curing method may be better suited for certain conditions. If precipitation is anticipated during the curing period, sealers and curing compounds usually need to be reapplied because water may wash out the product.

Several of the following recommendations are adapted from *ACI 301-16* and *ACI 308R-16 – Guide to External Curing of Concrete* and are presented here because of their relevance to this project *(16,21).* The two curing methods described should result in adequate durability performance of concrete, as long as the methods and procedures strictly follow the two ACI guides. The contractor should weigh the benefits and disadvantages of each method and submit a plan for approval by MassDOT.

1. **Curing concrete using saturated covers should be conducted in accordance with *ACI 308R-16*. If concrete is placed in cold weather conditions, curing should follow cold weather concreting practices (concrete protection, maintenance of minimum temperatures, and curing period) contained in *ACI 306R-16 – Guide to Cold Weather Concreting* and *ACI 306.1-90 – Standard Specification for Cold Weather Concreting*.** *ACI 308R-16* indicates that saturated covers have to be applied for a minimum of 7 days after placement to allow adequate cement hydration that will result in proper strength gain *(21).* Because of their slower rate of strength gain per *ACI 232.2R-18* and Sutter et al., 2014, respectively *(22,23),* concrete mixtures containing SCMs, particularly those containing Class F fly ash, should be cured for at least 10 days. Properly cured cement concrete mixtures develop approximately 70% of their design strength if curing is maintained for at least 7 days. Concrete mixtures containing SCMs require between 10 and 14 days to reach 70% of their design strength. Rate of strength gain is negatively affected at low curing temperatures. If temperatures are expected to fall below 40º F during the curing period, measures should be taken to maintain a minimum temperature of 50º F on the concrete surface to ensure the anticipated strength gain. In no case should water be allowed to freeze on the surface during the curing period.

2. **The use of concrete sealers and curing compounds satisfying *ASTM C1315* Type I (clear or translucent liquid membrane-forming sealant), Class A (non-yellowing curing compound) is acceptable for curing and may provide a means to facilitate field operations and mitigate issues with inadequate moisture content when using saturated covers** *(11).* If curing compounds satisfying *ASTM*

C309 (10) are used, a sealing compound may be required to prevent freezable water ingress into the concrete. Periodic resealing may be required prior to each winter season. The timing for application curing compounds as recommended by the manufacturer of the product must be strictly followed. Timing of application is particularly important with mixtures containing SCMs, which typically experience a decreased rate of bleed water migration. The potential for entrapping bleed water increases if sealing compounds are applied prematurely, thereby leading to the creation of a weak top layer in the concrete. In no case should the product be applied when bleed water is present on the concrete surface. Furthermore, reapplication of sealants to maintain an impervious surface of concrete might be required based on the product used. Reapplication of sealants should be done in accordance with the type of product and manufacturer recommendations, given the large variety of sealants and protective products in the market (refer to *ACI 515.2R-13 – Guide to Selecting Protective Treatments for Concrete (24)*).

3. **Cold Weather Concreting Practices. Follow cold weather concreting practices in accordance with *ACI 306R-16 – Guide to Cold Weather Concreting* and *ACI 306.1-90 – Standard Specification for Cold Weather Concreting* if low temperatures are expected after placement during the protection and curing period as defined in these guides.** Maintaining a proper temperature throughout the curing period and protecting concrete from low temperatures during the first several days after placement are fundamental practices to enable concrete to mature at a reasonable rate so that it can withstand the demands of freeze-thaw cycles. Section 3.6 of *ACI 308R-16* provides recommendations for cold-weather protection and curing of concrete *(21).* The guide recommends that if concrete will be critically saturated when exposed to freezing and thawing temperatures, protection and curing should be continued until a compressive in-place strength of approximately 4000 psi is reached. For resistance to deicer scaling, an in-place compressive strength of 4500 psi should be attained before such exposure is permitted. **In no case, however, should curing be stopped prior to the minimum curing periods recommended in point 1.**

4. **Hot Weather Concreting Practices. Follow hot weather concreting practices in accordance with *ACI 305.1-14 – Specification for Hot Weather Concreting* and *ACI 305R-20 – Guide to Hot Weather Concreting* if concrete is placed under hot weather conditions as defined in these guides** *(18,20)***.** This project did not evaluate the result of hot weather practices during construction of the sidewalks subject of this study. Sidewalks were not placed in hot weather conditions in this project. However, it is recommended that for sidewalks placed in hot weather conditions, the contractor should take proper measures to ensure adequate curing of concrete. Prevention of excessive surface water evaporation is perhaps one of the most critical aspects that needs to be followed.

9.3 Winter Treatment Operations

Sidewalks in this study were found to be particularly sensitive to scaling when conditions in the field exposed panels to chloride-laden water that remained on the surface of concrete for extended periods. This effect was particularly notable for sidewalk panels in Groups C, F, and I, which were on the eastern side of the site and where splash from snow/ice removal operations remained on the surface of the sidewalks. Other sidewalks at the site were cast following the pavement grade, which allowed melted snow to be washed off toward the lower portion of the site (east side). The following recommendations are made from these observations.

1. **Minimize exposure of sidewalks to deicing chemicals (chlorides) for as long as possible**. There does not seem to be a consensus on the specific period to avoid applying deicing chemicals to sidewalks after placement to prevent scaling. Some sources indicate that concrete should have matured enough to develop its design strength prior to application of deicing chemicals, while others call for a period of up to a year prior to application of salts (NRMCA, 1998 *(25)*; FHWA, 2015 *(26)*). It seems like the best practice would be to place sidewalks several months before the first application of deicing chemicals is expected (late summer). If this is not practical, the concrete mixture should be designed so that an in-place compressive strength of f'_c = 4500 psi has been reached prior to application of deicing chemicals. Following cold-weather procedures, particularly during the concrete protection and curing periods, is fundamental. If a curing and sealing compound is used, reapplication of the product should be evaluated based on the chemical composition of the sealant and be done in compliance with manufacturer recommendations. Furthermore, snow or ice containing deicing chemicals should be removed from the surface of sidewalks as soon as possible.

2. **Ensure that snow contaminated with chlorides is not allowed to remain on top of sidewalks following a storm event**. Snow and ice removal should be done as soon as practically possible. Surface scaling of concrete caused by a freezing water-chloride solution is a phenomenon that is not fully understood. Valenza and Scherer *(27,28)* concluded that ice that contracts on the imperfect surface of concrete as temperatures drop may create tensile stresses that could cause micro-cracks to propagate parallel to the surface. This mechanism, combined with the potential for a thermal gradient near the surface, may contribute to surface scaling, especially if freeze-thaw cycles are repeated. Increased damage may occur if temperatures fall below 14º F as indicated by Valenza and Scherer *(27,28)*.

3. **The use of two different deicing chemicals (NaCl and $MgCl_2$) in the field did not result in significant differences in scaling**. Results of the automated photogrammetry indicated that sidewalk panels subjected to NaCl splash but also treated with $MgCl_2$ performed slightly better than those treated only with NaCl. Given the limited nature of panels being studied under these conditions, no specific recommendation can be given here other than further studying the effect of combined deicing agents.

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11.0 Appendices

This report and the full version of the appendices can be found at: https://doi.org/10.7275/3nez-qd15