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# Construction and Materials Best Practices for Concrete Sidewalks: Phase II – Long-Term Performance and Hot-Weather Placement Effects

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16. Abstract

This report summarizes the investigation of construction practices and materials to develop durable concrete sidewalks which can resist scaling damage caused by exposure to freezing environment and deicer application. Over 16 months, a field study accompanied by laboratory testing was conducted to identify factors that affect the performance and durability of sidewalks.

The variables considered for the study are concrete mix design, placement and finishing practices, curing methods, and deicer application. The placement of the sidewalks took place in late July 2021, to investigate the impact of hot weather concreting practices on the performance of sidewalks. Forty-eight sidewalk panels were placed behind Robert Brack Structural Engineering Laboratory at University of Massachusetts Amherst (UMass). During the sidewalk placement, cylinders and rectangular prisms were placed for laboratory testing. Thirty-two rectangular prisms were subjected to same curing method as the corresponding sidewalks for scaling resistance test in laboratory via BNQ NQ 2621-900. The results of this study indicate that mixture design formulation, curing method, de-icing method, and temperature based concreting practices impact the performance of scaling in concrete sidewalks. Recommendations incorporating these variables are presented in this report with accompanying testing standards and procedures


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# Construction and Materials Best Practices for Concrete Sidewalks: Phase II – Long-Term Performance and Hot-Weather Placement Effects

Final Report


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# Disclaimer

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# Executive Summary

The research project of Construction and Materials Best Practices for Concrete Sidewalks: Phase II – Long-Term Performance and Hot-Weather Placement Effects is part of the ongoing study undertaken by Massachusetts Department of Transportation (MassDOT) Research Program and was funded with Federal Highway Administration (FHWA) State Planning and Research (SPR) funds.

This report summarizes the investigation of construction practices and materials to develop durable concrete sidewalks which can resist scaling damage caused by exposure to freezing environment and deicer application. Over 16 months, a field study accompanied by laboratory testing was conducted to identify factors that affect the performance and durability of sidewalks.

The variables considered for the study are concrete mix design, placement and finishing practices, curing methods, and deicer application. The mix designs considered of the study included 25% fly ash, 50% slag, and a mix with 100% cement. A total of five mix designs are developed for the study, two of the mixes were considered as poor mixes with no air entrainment and excess aggregates. Four curing methods were studied: moisture curing with saturated covers, curing with a chemical compound conforming to ASTM C1315, no curing, and a colloidal silica sealer. The placement of the sidewalks took place in late July 2021, to investigate the impact of hot weather concreting practices on the performance of sidewalks. Three deicer agents were considered for the study: sodium chloride (NaCl), magnesium chloride ($MgCl_2$), and blended brine (15% $MgCl_2$ + 85% NaCl). Forty-eight sidewalk panels were placed behind Robert Brack Structural Engineering Laboratory at University of Massachusetts Amherst (UMass). During the sidewalk placement, cylinders and rectangular prisms were placed for laboratory testing. Thirty-two rectangular prisms were subjected to same curing method as the corresponding sidewalks for scaling resistance test in laboratory via BNQ NQ 2621-900.

The results of this study indicate that mixture design formulation, curing method, de-icing method, and temperature based concreting practices impact the performance of scaling in concrete sidewalks. Recommendations incorporating these variables are presented in this report with accompanying testing standards and procedures.

Scaling on the top surface of concrete sidewalks exposed to de-icing chemicals is induced mainly by concrete tensile strength and the concretes mixture properties of the top 3 – 6 mm. A low water to cement ratio, high concrete strength before exposure to first freeze, and a large air void spacing factor is vital for adequate salt scaling resistance. The adherence to proper construction practices, material properties, and maintenance procedures should be monitored closely to attain salt scaling resistance. On comparison of this study with Phase I of the research project, it was concluded that sidewalks placed using hot weather concreting practices show better scaling performance than sidewalks placed following the cold weather concreting practices. The sidewalks form Phase I were placed in early November 2019 and were subjected to early freeze-thaw cycles. From the petrographic study it was determined that cores from Phase I exhibited a weak top layer while cores from Phase II did not, which made the sidewalks from Phase I more susceptible to scaling. It was also observed that the “poor” mixes from Phase II have outperformed the mix design from Phase I. A crucial conclusion from the comparison is that weather concreting practices also have an impact on scaling performance of concrete. The time between placement of sidewalks and exposure to first freezing temperatures has an impact on the scaling resistance of concrete. The longer the time, maturity of concrete increases and desired properties of concrete are achieved to resist scaling damage.

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# 1.0 Introduction

The Construction and Materials Best Practices of Concrete Sidewalks Phase II was a joint collaboration by the Massachusetts Department of Transportation (MassDOT) Research and Materials Laboratory, the University of Massachusetts Amherst (UMass), with the assistance of industry and contractor participants.

**Table 1.1 Participants of the study**

| Entity | Participants |
|---|---|
| MassDOT | Construction Section: District 2, Research and Materials Section |
| Academia | University of Massachusetts Amherst |
| Petrographic Examination Laboratory | Wiss, Janney, Elstner Associates, Inc. |
| Excavator | Baltazar Contractors |
| Core Drilling | Prime Drilling |
| Construction | Caracas Construction |
| Cement Concrete Producer | Construction Service (Readymix), Ralph Olds |
| Concrete Industry Participants | Construction Industries of Massachusetts, John Pourbaix<br>D.W. White Construction Inc, Jack Harney<br>Massachusetts Concrete and Aggregate Producers (MaCAPA), Craig Dauphinais<br>Lafarge/Holcim, Brian Barry |

Phase - I of the project *(1)* was concluded in April 2021. The project included six mix designs with varying amounts of fly ash (between 15% and 37%) and slag (between 25% and 50%), three curing methods, and two deicing agents. The curing methods investigated are no curing, saturated covers, and curing with a chemical compound. The placement of the sidewalk panels was done in early November 2019. The sidewalk panels were maintained and document throughout the winter. The field study included photogrammetric analysis of sidewalks and petrographic study of cores from all sidewalk panels. Laboratory experimentation included scaling resistance test via ASTM C672 *(2)*, fresh and hardened concrete properties tests and testing of aggregate system. The results of the study indicate that mix design, placement, finishing, curing practices, and quality control are factors that control the performance of sidewalks.

## 1.1 Problem Statement

Due to the rapid deterioration of concrete sidewalks caused by scaling statewide, it is urgent that concrete practices and materials be studied to determine their effect on the durability of concrete. The Commonwealth of Massachusetts experiences extreme winter weather conditions which has led to rapid degradation due to scaling of concrete sidewalks, leading to costly maintenance and reconstruction costs.

## 1.2 Objective

This research study is the second phase of a two-phase project to identify the causes that lead to surface scaling of concrete sidewalks after winter treatment. In Phase I, sidewalks were placed during the fall season to capture construction practices typical of cold weather. The research in Phase II primarily targets the effects of hot weather concreting procedures on scaling performance of concrete sidewalks after being exposed to winter environment and various treatment procedures that might lead to scaling. Other construction practices included in Phase II are mixture design and curing practices since these were identified as critical on performance of sidewalks studied during Phase I. Determining the best practices to limit the effects of winter weather conditions on concrete sidewalks maximizes the efficiency of the materials and other costs of construction while minimizing the need for maintenance and re-building costs that the State funds to maintain and rebuild.

## 1.3 Scaling

Scaling is a form of damage on the surface of concrete caused by cyclic freezing and thawing and exposure to deicing salts *(3)*. ACI 201.2R *Guide to Durable Concrete (4)* defines surface scaling as "loss of paste and mortar from the surface of concrete". Several theories have been proposed in the literature to identify scaling mechanisms , such as the hydraulic pressure theory *(5)*, osmotic pressure theory, and glue-spall theory *(3)*. In general, all these theories describe the scaling mechanism as caused by cracking and subsequent removal of a weak top layer at the surface due to a pressure build-up in the pores of the concrete near the surface. When water freezes, it expands about 9% in volume, which generates tensile stresses that may cause cracking when they exceed the tensile strength of concrete.  Diffusion of water between pores due to differences in salt concentration also creates osmotic pressure which leads to cracking and eventually scaling. Salt crystallization in pores causes pressures in the pore structure of concrete. There are several mechanisms that lead to pressure generation, which when combined with a weak top layer (low tensile strength) may cause cracking and subsequent scaling. Several factors lead to the formation of a weak top surface such as: mix design formulation, finishing practices, and curing method. These factors are discussed briefly in the subsequent sections, and in more detail in Chapter 2.

### 1.3.1 Mix Design Formulation

A lower w/cm ratio increases the salt scaling resistance of concrete. At low w/cm, due to minimal bleeding, the strength of the mass concrete is closer to the strength of the top surface *(6)*. Air entrainment in concrete improves the salt scaling resistance of concrete as it reduces the amount of bleeding. Also, a spacing factor below the critical spacing factor of 200 μm ensures a satisfactory scaling resistance in concrete *(7)*. The pressure build-up due to freezing and thawing can be relieved with a proper air-void system. The air content, spacing, and size of voids affect the resistance of concrete to freeze-thaw damage.

The use of supplementary cementitious materials can increase the scaling resistance of concrete when properly finished and cured. The air content and w/cm must be carefully selected when SCMs are used to avoid lowering the resistance of concrete against scaling. When fly ash and slag are incorporated in concrete, the permeability of concrete increases, which is directly related to the durability of concrete.

### 1.3.2 Finishing Practices

The air-void system of fresh concrete and hardened concrete is usually different. Mixing, placing, and finishing of fresh concrete can alter the entrained air-void system. Over finishing can result in fewer and larger air voids on the top surface of the concrete, which is susceptible to scaling. When finishing is done before bleed water disappears, a weak top layer is formed on the surface. Finishing with bleed water decreases the air content and increases the w/cm and porosity in the top layer, which results in lower resistance to freezing and thawing.

### 1.3.3 Curing

Curing ensures the hydration process in concrete and leads to the achievement of desirable properties. Proper curing increases the resistance of concrete against freezing and thawing. The duration of curing and the method of curing affect the scaling resistance. When SCMs are utilized, the length of curing should be adequate as the early strength of fly ash and slag are low. Improper curing will lead to a weak top layer, drying shrinkage cracks, and reduced strength.

The time of application of curing compound is also important as it can result in trapped bleed water under membrane which will lead to a weak surface layer. It is evident that early application of curing compounds will result in poor formation and gaps in membrane because the density of curing compound emulsions is close to density of the bleed water *(8*.

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# 2.0 Scope

This project was undertaken to identify and evaluate best materials and practices for concrete sidewalks. The project, to date, has been divided into two phases. Phase I of the project focused on cold weather concreting and was completed in 2020 *(1)*.This report describes Phase II of the project, which focuses on hot weather concreting. Placement of concrete sidewalks took place in late July 2021, intentionally during hot days, to examine the effect of hot weather concreting practices on the scaling performance of sidewalks. By placing sidewalks in the summer, a significant amount of time elapsed allowing concrete to cure and gain strength before the first freeze in winter. Hot weather concreting practices were followed, as described by ACI 305R-20 Guide to Hot Weather Concreting *(9)*. The project focuses on the effect of curing methods, placement and finishing practices and deicer application on durability of concrete sidewalks. These factors were evaluated by conducting field and laboratory investigations, complemented by computer-based photogrammetric analysis. To assess the field performance, 48 concrete sidewalk panels were placed at the Robert Brack Structural Testing Facility (the Brack Laboratory) at UMass Amherst and monitored between July 2021 and July 2022. These panels varied, with five concrete mix designs, three deicing methods employed through the winter season, and four curing and sealing methods used in placement. The sidewalk panels were approximately 6 ft. long, 4 ft. wide and 6 in. deep. Several companies and laboratories contributed materials and services to the project; they are listed in Table 2.1.

Sidewalks were placed according to the schematic in Figure 2.1. Sidewalks were placed in three rows to allow different winter treatment procedures to be examined. Panels A, B and C were treated using sodium chloride (NaCl) as needed before winter events. Panels D, E, and F were treated using magnesium chloride (MgCl2). Panels G, H, and I were treated spraying a blended brine solution (85% NaCl + 15% $MgCl_2$) on the top surface of snow after winter storms. Winter treatment details used in the different sidewalk groups are provided in Section 2.5.1 dates when winter treatment was conducted.

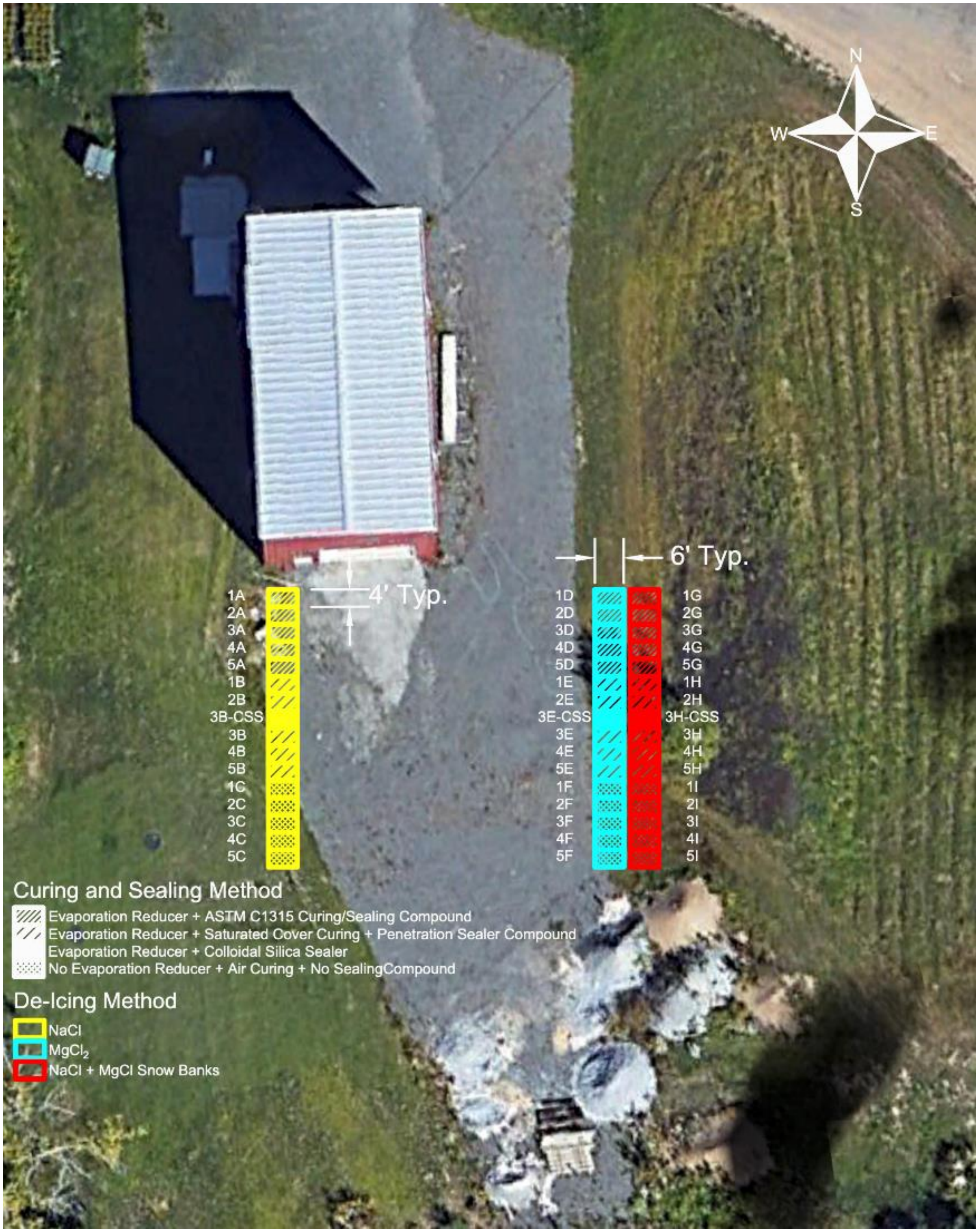



**Figure 2.1 Layout of sidewalks on south side of Robert Brack Structural Testing Laboratory at UMass Amherst**

## 2.1 Mix Design

The ready mix concrete supplier, Construction Service, developed five concrete mix design for the research project, which were approved by MassDOT. Key differences in concrete mix designs are presented here and further details of the mix designs are presented in Chapter 3. Concrete Mix 1 has 25% of fly ash replacement, 7% air entrainment, and 0.45 w/cm. Concrete Mix 2 has a 50% of slag replacement, 7% air entrainment, and 0.45 w/cm. Concrete Mix 3 is a 100% cement mixture with 0% supplementary cementitious materials (SCMs), 7% air entrainment, and 0.45 w/cm. Concrete Mix 4 has a 25% of fly ash replacement, no air entrainment, and 0.5 w/cm, and poor gradation due to excessive amount of fine aggregate. Concrete Mix 5 has a 50% SCM of slag, no air entrainment, 0.35 w/cm, and poor gradation due to excessive amount of coarse aggregate. The concrete mix proportions are summarized in Table 2.2. The w/cm ratio in the Table 2.1 are based on the batch tickets and include the water added on site and in transit.

**Table 2.1 Summary of mix design formulations**

| Mix Design No. | SCM Type | SCM (%) | Aggregates Fine (%) | Aggregates Coarse (%) | w/cm |
|---|---|---|---|---|---|
| Mix 1 | Fly Ash | 25 | 41.5 | 58.4 | 0.41 |
| Mix 2 | Slag | 49.7 | 42.0 | 57.9 | 0.43 |
| Mix 3-A | No SCM | 0 | 41.6 | 58.4 | 0.42 |
| Mix 3-B | No SCM | 0 | 32.3 | 67.7 | 0.44 |
| Mix 4 | Fly Ash | 25 | 62.1 | 37.9 | 0.51 |
| Mix 5 | Slag | 50.2 | 37.5 | 62.5 | 0.35 |

## 2.2 Pre-Placement and Placement Practices

This section is a summary of best practices for pre-placement and placement practices from the MassDOT Standard Specification for Highways and Bridges *(10)*,and will be compared to with the in-situ placement practices performed in this study in Section 5.1.

### 2.2.1 Pre-Placement Practices

The area should be excavated per *MassDOT specifications*, "Subsection 120: Excavation" before placement of subbase and subgrade.

The subgrade shall meet the requirements of *MassDOT specifications*, "Subsection 170: Grading". The subgrade area shall be placed parallel to the surface of the sidewalks. Any depressions in the subgrade shall be filled with suitable materials. The area is then compacted and graded until the subgrade does not require additional compaction and the surface is smooth.

A gravel subbase shall be placed on the subgrade. After compaction, the subbase shall have at least 8 inches of thickness. After placing the gravel, any high areas shall be trimmed, and low areas shall be filled with materials and compacted. Before placement of concrete, the subbase shall be checked to ensure the required grade and cross-section.

The depth of the form shall equal the thickness of the concrete sidewalk. The forms shall not have broken top surfaces or be bent or twisted. The forms shall be staked and checked for correct line and grade before concrete placement, and any disturbances must be corrected. The forms shall be cleaned from dirt or mortar and then oiled before placing concrete.

### 2.2.2 Placement Practices

Concrete sidewalks must be placed in accordance with *MassDOT Specifications,* "Subsection 701.41: Cement Concrete Sidewalks, Pedestrian Curb Ramps, and Driveways" and best practices as mentioned by National Ready Mixed Concrete Association (NMRCA).
Concrete shall not be placed on an excessively wet or frozen subbase. The concrete shall be spread to avoid segregation, and after consolidation, the thickness of the sidewalk shall be 6 inches.

The placement of concrete slabs shall be done in alternate slabs of 30 ft long, which are separated by expansion joints filler of ½ inch thickness. The concrete sidewalks shall be uniformly scored into blocks with an area not greater than 36 $ft^2$. The depth of the scoring must be at least ½ inch, and the width shall not be more than ½ inch.

## 2.3 Finishing Practices

### 2.3.1 Finishing Practices

The concrete sidewalks were finished following *MassDOT Specifications* and *National Ready Mix Concrete Associations (NRMCA)* Best Practices.

The finishing shall be completed by skilled cement finishers. Using bull float, darby, or highway straightedge as flat as possible, smooth the concrete before bleed water emerges. Pause until bleeding finishes and water sheen clears off the surface. For hot, dry, and windy weather placement, cover the concrete surface to prevent evaporation. Surface finishing may continue when a person standing on the slab surface leaves a footprint impression more than 1/8$^{th}$ inch (3mm) and less than ¼ inch (6mm).

When edging, use small masons trowels to spade edge and use edging tool to achieve round edges. When jointing, using jointing tool with length equal to 1/4$^{th}$ panel depth and horizontally use level lumber to assist. Ornamental groves can be created using shallow-bit groover. Enclose larger aggregated and further level using hand float or machine.

**Figure 2.2 Worker trowels sidewalk panel following MassDOT specifications**

Use a push-broom with fine or coarse bristles after floating to texture surface and construct slip-resistant panel finish. Subsequent to finishing, begin curing process. Following provisions *476.71: Curing* and *476.74 Protection of Pavement*, sidewalks should be maintained saturated, nonoperational, and sheltered from weather for at minimum 3 days.

### 2.3.2 Prohibited Finishing Practices

Following *NRMCA Best Practices,* wait until float water is removed from the surface before floating concrete. Do not trowel on air-entrained concrete unless required, if required then be alert on scheduling the finishing phase. Dark trowel burns can be produced from overdone troweling and an incorrect tilt creates unsatisfactory texture. In the finishing phase do not spray water or dust cement onto the surface.

## 2.4 Curing Methods

The concrete sidewalks were subjected to moisture curing with saturated covers, curing with a chemical compound, and no curing to evaluate their effect on the performance of sidewalks. The curing methods in this study that were followed are in accordance with ACI 305R *Guide to Hot Weather Concreting (9*) and ACI 308R *Guide to External Curing of Concrete (11).* As these guides indicate, additional care must be used for curing when concrete is placed in hot weather. Several issues corresponding to hot weather concreting are evaporation of moisture from the surface and rapid concrete setting time, leading to drying shrinkage cracking. Thermal shrinkage cracking also occurs due to extreme ambient temperature differences during day and night. According to ACI 308R, evaporation reducers must be applied to prevent loss of water and allow concrete to reach its potential strength. Initial curing methods such as evaporation reducers should be used to reduce the rate of evaporation. When liquid evaporation reducers are applied, the chemical forms a thin film on top of the concrete surface and locks in the bleed water, which rises to the top of the concrete surface. These are mainly required when the evaporation rate exceeds the bleed water rate. The following sections describe best practices for various curing methods.

### 2.4.1 Saturated Cover Curing

#### 2.4.1.1 Materials

The saturated cover (burlap) used for curing should be free of harmful, soluble materials and substances which may cause discoloration. The burlap must be rinsed with clean water and made sure to be free of fungus or bacteria. The burlap being reused must be washed and made sure not to carry over any substances from previous use or storage. The material used as saturated cover must have enough thickness to hold moisture, be stable against winds and reduce the necessity for rewetting frequently. Continuous wetting and drying of burlap should be avoided as it leads to pattern cracking. The water utilized for curing should be free of harmful chemicals that may deteriorate concrete, such as chlorides. The curing water should not have organics materials that can cause stains or discoloration of concrete surfaces. The temperature of the curing water should not be more than 20 ºC to avoid thermal shock in concrete, which can lead to shrinkage cracking.

#### 2.4.1.2 Application

The burlap should always lay flat and remain in contact with the sidewalk's surface. The saturated cover strips should be allowed to lap at half-widths to aid water retention and avoid displacement of covers due to winds or rain. The final curing should be followed by the initial curing method. The burlap should be moist throughout the duration of final curing and not be allowed to dry. Drying of burlap will result in burlap absorbing water from the concrete surface, which results in discoloration on the surface due to efflorescence. Entrapped air should be avoided by laying burlap flat without entrapped air to prevent efflorescence. The final curing method must be done when the concrete surface is sufficiently hardened but not delayed to a time when it is detrimental to concrete. The final saturated cover curing must be done so that concrete gains its desired properties by maintaining an appropriate moisture content and temperature. The internal temperature of the concrete should not exceed 70 ºC.

### 2.4.2 Liquid Membrane-forming Compound Curing

#### 2.4.2.1 Materials

The liquid membrane-forming compound used for curing should meet the requirements of *ASTM C1315 Liquid Membrane-Forming Compounds Having Special Properties for Curing and Sealing Concrete*. The ASTM C1315 compounds aid in moisture-retention and have properties such as acid resistance, alkali resistance, and resistance against UV radiation. In addition, these compounds form a film and restrict moisture loss from concrete, allowing concrete to attain its desired properties. For this study, an ASTM C1315 Type I compound has been utilized. The curing and sealing compound utilized has a red fugitive dye which is not typical of this product.

#### 2.4.2.2 Application

The application rate of ASTM C1315 Type I/Type II compounds is 300 $ft^2$/gal, which complies with laboratory testing in accordance with ASTM C156 on smooth and even concrete surface. However, the compound coverage rate must be greater based on the texture of the concrete surface on the field. When the surface is deeply textured, at least two applications are needed to ensure desired moisture retention rate and full coverage of the surface. The rate of application may also be influenced by the rate of evaporation on the field.

The curing compounds must be stirred before application. The moisture-retention rates of compounds vary by products. Therefore, the application rate must comply with the manufacturer's recommendation. The compound must be applied uniformly twice, the second layer perpendicularly crossing the first, ensuring complete coverage. The application can be done by hand or sprayer, depending upon the size of the job. A power sprayer is preferred for a large area to maintain uniformity and speed. The pressure from the nozzle of the power sprayer must be between 0.2 to 0.7 MPa. A brush or paint roller can be used to apply the compound in smaller areas.

After final finishing, the compound must be applied as soon as the surface water sheen disappears. To avoid surface drying and water loss, the application of curing compounds must not be delayed. If surface drying occurs, the liquid curing compound will be absorbed by concrete and not form a membrane.

The surface may appear dry when the evaporation rate is significantly higher than the bleed rate. In this case, the finishing and application of the curing compound will affect the concrete as the bleed water will get trapped under the concrete surface resulting in a weak top layer, map cracking of membrane film, and loss of moisture-retention capabilities. To avoid such risk, a test must be done by placing a 450 mm square plastic cover on an unfinished and uncured surface to avoid evaporation and check for accumulation of bleed water.

In hot weather concrete placement, moisture loss from the concrete surface must be prevented. In such cases, the surface must be kept moist to achieve a uniformly damped surface that is free of water on the surface until the application of the curing compound. When the surface is damp, the curing compound will not be absorbed by concrete, and a membrane film will be formed as required.

### 2.4.3 No Curing

The concrete panels are not subjected to evaporation reducers, curing compounds, or saturated cover. Instead, these panels are subjected to ambient weather conditions.

## 2.5 Application of De-Icing Salts

### 2.5.1 Winter Maintenance

Procedures consistent with the MassDOT procedures were followed to maintain and treat sidewalks throughout the winter season. Sidewalks were generally treated within 24 hours before a snow event. If the event produced 2 in. of snow or more, sidewalks were treated within a 24-hour window before and after the event.

**Table 2.2 Pre and post treatment application dates**

| Treatment | Panel Groups | December 2021 | January 2022 | February 2022 | March 2022 |
|---|---|---|---|---|---|
| Pre-Treatments | D, E, and F ($MgCl_2$) | 7, 17, 23,27 | 4, 6, 8, 16, 24, 28 | 3, 6, 18, 24 | 9,11 |
| Pre-Treatments | G, H, and I (Blended Brine) | - | 4, 6, 8, 16, 24, 28 | 3, 6, 18, 24 | 9,11 |
| Post-Treatments | Panel Groups | December 2021 | January 2022 | February 2022 | March 2022 |
| Post-Treatments | A, B, and C (NaCl) | 9, 24 | 7, 25, 30 | 5 | 10,13 |
| Post-Treatments | D, E, and F ($MgCl_2$) | 9, 24 | 7, 25, 30 | 5 | 10,13 |
| Post-Treatments | G, H, and I (Blended Brine) | | 7, 25, | 5 | 10,13 |
| Snow Event Days | | 8. 24- | 7, 17, 29 | 4 | 9, 12 |

Table 2.2 shows dates on which pre-treatments and post-treatment applications occurred. Pre-treatment is applied more frequently than post treatment due to weather forecasting accuracy and ease of access to the sidewalk panels prior to a weather event. Blended brine treatment started in January due to delays in acquiring the deicer.

### 2.5.2 Sodium Chloride Application

Application of sodium chloride was only conducted after snow events. Panel Groups A through C did not receive pre-treatment. In post treatment, Panel Groups A through C were shoveled and receive approximately 10 lbs of rock salt applied evenly by hand. It was determined that approximately 10 lbs of rock salt is sufficient to uniformly cover the panel section.

### 2.5.3 Magnesium Chloride Application

Following common field practice, between 0.1 to 0.2 gal/1000 sq-ft of deicer was applied. A 300 ml/384 sq ft rate of application was adopted based on the total area of sidewalks. Using a common weed sprayer, the nozzle spray rate was determined, and the 300 ml of liquid sprayed in approximately 1 minute. Panel Groups D through F received 300 ml of Magnesium Chloride ($MgCl_2$) as both pre-treatment and post treatment. Snow was shoveled prior to application of post treatment $MgCl_2$.

### 2.5.4 Brine (85% NaCl and 15% $MgCl_2$) Application

The same quantity and application as $MgCl_2$ were followed for Blended brine. This resulted in our procedure to evenly spray a row of panels for 1 minute. Panel Groups G through I received 300 ml of Blended Brine (85% NaCl and 15% $MgCl_2$) as both pre-treatment and post treatment. Snow was not shoveled off prior to application, and the blended brine was sprayed directly on the snow. The effect of spraying Blended Brine on the surface of the snow is to replicate the accumulation of snow on corners and edges of sidewalk panels after clearing. The snow mixes with the deicers but ultimately remains on the panels. Panels were not shoveled unless required to take progress photos of the panels for analysis.

(a) Removal of snow on panels D, E, and F which are subjected to $MgCl_2$ (Right). Panels G, H, and I treated with blended brine on accumulated snow (Left) on February 26th, 2022.

(b) Panels D, E, F, G, H, and I covered with snow on December 24th, 2021.

**Figure 2.3 Treated panels—$MgCl_2$ and brine**

(a) Panels A, B, and C treated with NaCl after removal of snow on January 30th, 2022.

(b) Panels A, B, and C covered with 6" of snow on February 26th, 2022

**Figure 2.4 Treated panels—NaCl**

# 3.0 Mix Design Formulation

The five concrete mix design formulations used in the study are presented in this chapter. The mix quantities reported in the chapter are in accordance with the batch tickets which represent the mixes delivered at site. The water content included as delivered and additional water added on site and in transit. The sources and quantities of materials utilized for the study are reported in Tables 3.1 through 3.4.

**Table 3.1 Aggregates**

| Materials | Manufacturer | Location | Description | AASHTO |
|---|---|---|---|---|
| Fine | Delta Sand & Gravel | Sunderland, MA | Normal Weight | M 6 |
| ¾ in | J S Lane | Amherst, MA | Normal Weight - 67 | M 80 |

**Table 3.2 Cement and Supplementary Cementitious Materials**

| Materials | Manufacturer | Location | Type | Description | AASHTO |
|---|---|---|---|---|---|
| Cement | Lafarge | St Constant, QC | I/II | General / Mod Sulfate | M 85 |
| Fly Ash | Ciment Quebec | 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**

| Materials | Manufacturer | Product | Type | Description | AASHTO |
|---|---|---|---|---|---|
| AD1 | Master Builders Solutions | Master Air AE 200 | P-AEA | Air Entraining | M 154 |
| AD2 | Master Builders Solutions | Master Glenium 7500 | A | Water Reducing | M 194 |
| AD3 | Master Builders Solutions | Master Sure Z 60 | S-WRK | Workability Retaining | M 194 |

**Table 3.4 Mix design formulation as per batch tickets (per $yd^3$)**

| **Mix No.** | **3/4 TR** lbs. | **3/4 TRAP** lbs. | **Fine** lbs. | **Cement** lbs. | **Fly Ash** lbs. | **Slag** lbs. | **Water**[1] gal. | **AD 1** oz. | **AD 2** oz. | **AD 3** oz. |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 782 | 1008 | 1270 | 468 | 156 | - | 30.6 | 3.06 | 22.2 | 12.6 |
| 2 | 772 | 1006 | 1290 | 319 | - | 316 | 33 | 3.00 | 22.2 | 12.6 |
| 3A | 796 | 1002 | 1280 | 628 | - | - | 31.4 | 2.52 | 21.80 | 12.4 |
| 3B | 798 | 1290 | 997 | 614 | - | - | 32.7 | 2.50 | 21.83 | 12.3 |
| 4 | 600 | 598 | 1966 | 456 | 152 | - | 37.4 | - | 12.2 | - |
| 5 | 248 | 1000 | 748 | 401 | - | 404 | 34 | - | 48.6 | - |

Note:
[1] The water is the total amount of water including water added in transit and on site.

## 3.1 Combined Aggregate System

As per AASHTO T 27 *Sieve Analysis of Fine and Coarse Aggregates*, the aggregates quantities are tested and tabulated in Table 3.5.

**Table 3.5 Combined aggregate system particle size distribution per AASHTO T 27**

| Property | Mix 1 % by mass retained on each sieve | Mix 2 % by mass retained on each sieve | Mix 3 % by mass retained on each sieve | Mix 4 % by mass retained on each sieve | Mix 5 % by mass retained on each sieve |
|---|---|---|---|---|---|
| 1 1/2 in. | 100 | 100 | 100 | 100 | 100 |
| 1 in. | 100 | 100 | 100 | 100 | 100 |
| 3/4 in. | 98.9 | 98.9 | 98.9 | 99.1 | 98.7 |
| 1/2 in. | 80.4 | 80.5 | 80.4 | 84.5 | 77.1 |
| 3/8 in. | 60.8 | 62.0 | 61.8 | 69.8 | 55.3 |
| No. 4 | 44.9 | 45.1 | 44.8 | 56.4 | 35.4 |
| No. 8 | 39.1 | 39.4 | 39.1 | 50.2 | 29.9 |
| No. 16 | 32.9 | 33.2 | 32.9 | 42.3 | 25.2 |
| No. 30 | 20.3 | 20.4 | 20.2 | 25.9 | 15.7 |
| No. 50 | 7.5 | 7.5 | 7.5 | 9.3 | 6.0 |
| No. 100 | 2.9 | 2.9 | 2.8 | 3.3 | 2.4 |
| No. 200 | 1.6 | 1.6 | 1.6 | 1.8 | 1.5 |

The Tarantula Curve, Shilstone Workability-Coarseness Chart *(11)*and void content are utilized to analyze the combined aggregate system for each mix design. The tables and figures in Sections 3.1.1 and 3.1.2 are computed by cement concrete producer (Construction Service-Springfield, MA) using spreadsheets provided by MassDOT for concrete mixture approval. This is conducted to ensure that the aggregate system satisfies the requirements for a smooth aggregate grading curve.

### 3.1.1 Tarantula Curve

A tarantula curve *(11)* provides recommended limits for different sizes of aggregates retained on a sieve to achieve concrete with adequate workability. It provides an approach to proportioning aggregates for a workable concrete mixture. The total amount of coarse aggregates retained on sieves No. 8 to No. 30 must be >15%. The total retained volume of coarse aggregates (No. 8 to 30) affects the cohesive properties of concrete and its resistance against segregation and edge slump. The total amount of fine aggregates retained on sieves No. 30 to No. 200 must be within the limits of 25% and 40% for flowable concrete applications. This total volume of fine aggregates affects the finishability of the concrete mix. Table 3.6 and Figure 3.1 provide the ideal Tarantula curve limits along with example fine and coarse aggregates that lie within recommended limits. In this example, aggregates are likely to produce cement concrete with good workability, finishability, and cohesion.

**Table 3.6 Particle size distribution per tarantula curve recommended limits**

| Sieve Opening | Passing % by Mass | Retained % by mass | Retained Ranges % by mass | Retained Ranges % by mass | Retained Ranges % by mass |
|---|---|---|---|---|---|
| 1 ½ 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 20-10 | - |
| No. 16 | 31 | 6 | 0-12 | Coarse Sand 20-10 | - |
| No. 30 | 18 | 13 | 4-20 | Coarse Sand 20-10 | Fine Sand 25-40 |
| No. 50 | 5 | 13 | 4-20 | - | Fine Sand 25-40 |
| No. 100 | 0 | 5 | 0-10 | - | Fine Sand 25-40 |
| No. 200 | 0 | 0 | 0-1 | - | Fine Sand 25-40 |

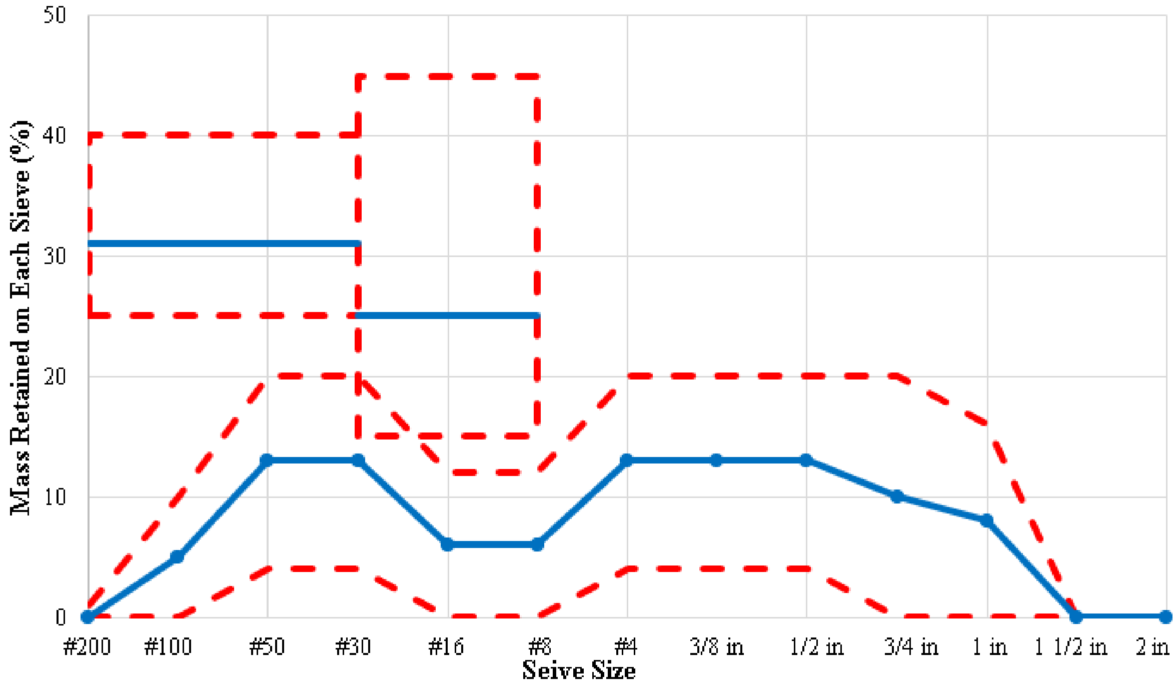



**Figure 3.1 Particle size distribution per tarantula curve recommended limits**

Table 3.7 and Figure 3.2 provide the combined aggregate system for each mix design formulation used in this project compared with the recommended limits of the tarantula curve. From the Tarantula curve results, it must be noted that Mix 4 and 5 have excessive fine and excessive coarse aggregates, respectively.

**Table 3.7 Particle size distribution of each mix design formulation**

| Property | Mix 1 %by mass retained | Mix 2 %by mass retained | Mix 3 %by mass retained | Mix 4 %by mass retained | Mix 5 %by mass retained | Criteria %by mass retained | Criteria %by mass retained | Criteria %by mass retained |
|---|---|---|---|---|---|---|---|---|
| 1 1/2 in. | 0 | 0 | 0 | 0 | 0 | - | - | - |
| 1 in. | 0 | 0 | 0 | 0 | 0 | 0-16 | - | - |
| 3/4 in. | 1.1 | 1.1 | 1.1 | 0.9 | 1.3 | 0-20 | - | - |
| 1/2 in. | 18.5 | 18.4 | 18.5 | 14.6 | 21.6* | 4-20 | - | - |
| 3/8 in. | 18.6 | 18.5 | 18.6 | 14.7 | 21.8 | 4-20 | - | - |
| No. 4 | 17.0 | 16.9 | 17.0 | 13.4 | 19.9 | 4-20 | - | - |
| No. 8 | 5.8 | 5.8 | 5.8 | 6.2 | 5.4 | 0-12 | Coarse sand 20-40 | - |
| No. 16 | 6.2 | 6.2 | 6.2 | 7.9 | 4.7 | 0-12 | Coarse sand 20-40 | - |
| No. 30 | 12.7 | 12.8 | 12.7 | 16.4 | 9.6 | 4-20 | Coarse sand 20-40 | Fine Sand 25 -40 |
| No. 50 | 12.8 | 12.9 | 12.8 | 16.6 | 9.6 | 4-20 | - | Fine Sand 25 -40 |
| No. 100 | 4.6 | 4.7 | 4.6 | 6.0 | 3.6 | 0-10 | - | Fine Sand 25 -40 |
| No. 200 | 1.2 | 1.3 | 1.2 | 1.5 | 1.0 | 0-1 | - | Fine Sand 25 -40 |

Note:
*Orange cells indicate values exceeding the given criteria from the tarantula curve approach

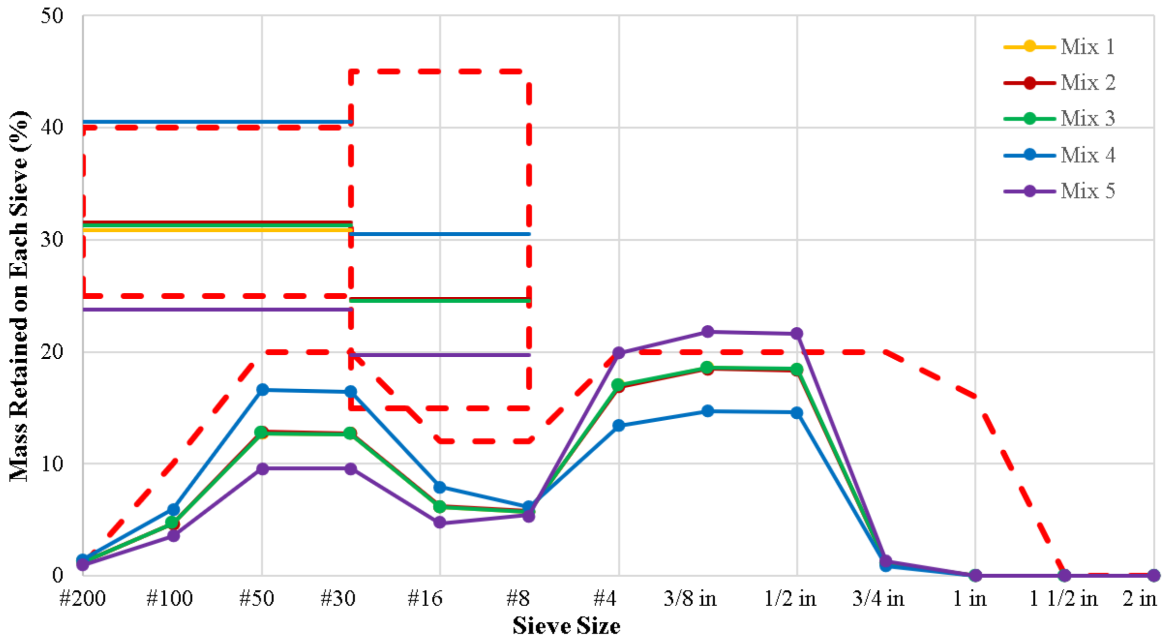


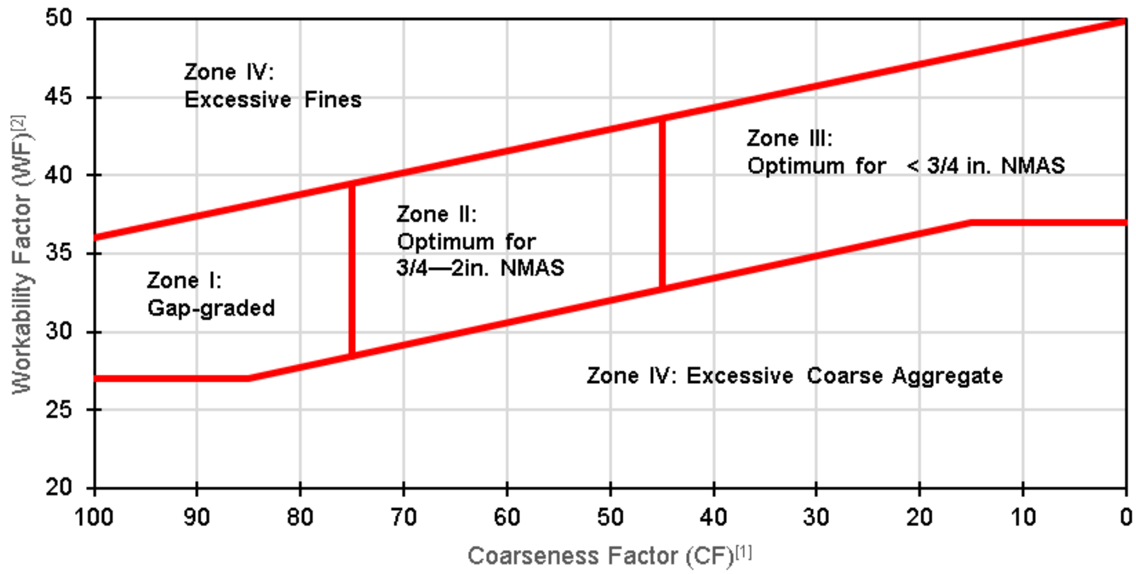

**Figure 3.2 Tarantula curve results**

### 3.1.2 Shilstone Workability-Coarseness Chart

The Shilstone Workability-Coarseness Chart (REF) is used to determine the optimized workability and coarseness factors for a given particle size. Identifying mixture designs with optimum Shilstone Workability and Coarseness factors can prevent undesired properties such as cracking, blistering, spalling, and scaling as shown in Table 3.8.

**Table 3.8 Shilstone workability-coarseness zones**

| Zone | Cause | Property |
|---|---|---|
| I | Gap-graded | Deficiency in intermediate particles; Non-cohesive; High potential for segregation during placement and consolidation; Cracking, blistering, spalling, and scaling |
| II | Optimum for NMAS 3/4 in. - 2 in. | Optimized workability factor and coarseness factor |
| III | Optimum for NMAS < 3/4 in. | Optimized workability factor and coarseness factor |
| IV | Excessive Fines | Sticky; High potential for segregation during consolidation and finishing; Variable strength, high shrinkage, cracking, curling, spalling, and scaling |
| V | Excessive Coarse Aggregate | Rocky; Lacking plasticity |




Notes:

[1] The Coarseness Factor (CF) is calculated through the equation: (CF) = (Q/R)*100
Where Q is cumulative % retained on 3/8 in. sieve, and R is cumulative % retained on no. 8 sieve.
[2] The Workability Factor (WF) is calculated through the equation: (WF) = W + (2.5(C-564)/94)
Where W is the % passing no. 8 sieve, and C is cementitious material content (lb/yd$^3$)

**Figure 3.3 Shilstone Workability-Coarseness Chart**

The Coarseness Factors and Workability Factors are calculated from sieve analysis for each mix and presented below in Table 3.9. The zone where aggregates for this project lie is determined by plotting the CF and WF on the Shilstone Chart (Figure 3.4).

**Table 3.9 Shilstone Workability-Coarseness Results**

| Mix | CF | WF | Zone |
|---|---|---|---|
| 1 | 62.66 | 40.86 | II |
| 2 | 62.62 | 41.13 | II |
| 3 | 62.67 | 40.56 | II |
| 4 | 60.63 | 51.47 | IV |
| 5 | 63.84 | 36.51 | II |

Mixture Designs 1, 2, 3 and 5 are optimum and conform to the Shilstone Workability-Coarseness criteria. Highlighted is Mixture 4 which exceeds criteria for excessive fine material.

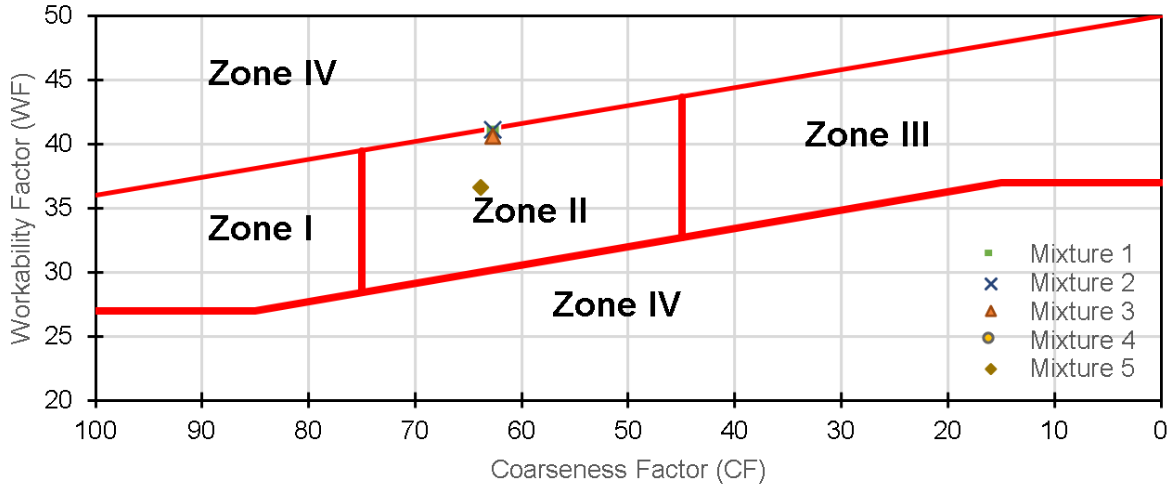



**Figure 3.4 Shilstone workability-coarseness chart for each mix design formulation**

Figure 3.4 displays the mixtures plotted on the Shilstone chart according to their calculated Coarseness Factor and Workability Factor. Mixture 4 has a workability factor > 50 and is off the standard Shilstone figure plot.

### 3.1.3 Void Content

The void content is used to determine the required water to cementitious materials (w/cm) ratio in the mixture design. The w/cm ratio is strongly related to durability and strength of concrete. The w/cm ratio effects workability, particle binding, and pore space. Void content is used in determining paste content to void content (PC/VC) and excessive paste content (EPC), both of which are discussed in Section 3.10.
The void content can be determined through this set of calculations, where SG = Specific Gravity, W = Weight (lbs.), V = volume (cfs), D = Density (pcf), UW = Unit Weight (pcf), and VC = Void Content (%).

$$V_{cement} = \frac{W_{cement}}{(SG_{cement} * D_{water})}$$

$$V_{scm} = \frac{W_{scm}}{(SG_{scm} * D_{water})}$$

$$V_{water} = \frac{V_{water}\ in\ gal.}{7.48\ gal. per\ cfs}$$

Volume of water includes water from mixing water, admixture liquid, and condensation from aggregate.

$$V_{coarse} = \frac{W_{coarse}}{(SG_{coarse} * D_{water})}$$

$$V_{fine} = \frac{W_{fine}}{(SG_{fine} * D_{water})}$$

$$V_{air} = 27\ cf * V_{air}\ (\%)$$

$$V_{yield} \quad = V_{cement} + V_{scm} + V_{water} + V_{coarse} + V_{fine} + V_{air}$$

Volume of yield does not include admixture volume because it is insignificant in the volume of yield calculation.

$$VC_{coarse} = \frac{(SG_{coarse} * D_{water}) - UW_{coarse}}{SG_{coarse} * D_{water}}$$

$$VC_{fine} = \frac{\left(SG_{fine} * D_{water}\right) - UW_{fine}}{SG_{fine} * D_{water}}$$

$$VC_{aggregate} = \left(\frac{V_{coarse}}{V_{coarse} + V_{fine}}\right) * VC_{coarse} + \left(\frac{V_{fine}}{V_{coarse} + V_{fine}}\right) * VC_{fine}$$

$$eq. 3.1: VC_{coarse} = VC_{aggregate} * \frac{V_{coarse} + V_{fine}}{V_{yield}}$$

The aggregate void content results for each mixture design used in this project shown are listed in Table 3.10.

**Table 3.10 Aggregate void content results**

| Property | Mix 1 | Mix 2 | Mix 3 | Mix 4 | Mix 5 |
|---|---|---|---|---|---|
| Aggregate Void Content (%) | 24.1 | 24.2 | 24.4 | 24.7 | 24.7 |

## 3.2 Paste System

The water-cementitious ratio, supplementary cementitious material content, chemical admixtures, and paste content are identified for paste system of each mix design formulation.

### 3.2.1 Water-Cementitious Ratio

The water-cementitious ratio is related to the strength and durability of concrete. If the amount of freezable water available in concrete is low, the damage from freezing and thawing can be reduced. By lowering the w/cm, the amount of freezable water can be decreased. In accordance with ACI 201.2R *Guide to Durable Concrete,* “Chapter 4 – Freezing and Thawing of Concrete”, the freezing-and-thawing exposure class for a sidewalk in F3- Very severe as the sidewalks will be exposed to freeze and thaw conditions as well as deicing chemicals. A maximum of 0.45 w/cm ratio is recommended for the F3 exposure class per ACI 201.2R. ACI 318-19 also recommends a w/cm ratio less than or equal to 0.45 for adequate freeze-thaw durability. Table 3.11 shows the w/cm ratio of all mix design formulations compared with the criteria. The w/cm ratio is computed by taking the ratio of total water content by mass and total cement plus SCM content by mass.

**Table 3.11 Water-cementitious material ratio results**

| Property | Mix 1 | Mix 2 | Mix 3A | Mix 3B | Mix 4 | Mix 5 | Criteria |
|---|---|---|---|---|---|---|---|
| Water in the mix (including addition on field and transit) | 30.6 | 33 | 31.4 | 32.7 | 37.4 | 34 | - |
| w/cm ratio | 0.41 | 0.43 | 0.42 | 0.44 | 0.51 | 0.35 | ≤ 0.45 |

### 3.2.2 Supplementary Cementitious Materials

Supplementary cementitious materials (SCM) are widely used to improve selected properties of concrete. The use of SCMs affects both fresh and hardened properties of concrete. When SCMs such as fly ash and slag are incorporated in mix design formulation, less water is required to achieve workability than mixes constructed with cement only, setting time is delayed, early after placement strength is lesser but later strengths are higher, permeability is reduced, and resistance to chloride ion penetration is increased. Due to these changes, the placement, finishing, and curing practices might be slightly altered when utilizing SCMs. Improper finishing and curing practices of concrete with SCMs will decrease the resistance to damage from freezing and thawing and deicers exposure.

Table 3.12 lists the maximum permitted SCMs content in mix design per ACI 201.2R. The limits given for exposure class F3 with hand-finished surfaces. When high volumes of SCMs replace cement, care must be taken to adequately cure the concrete to achieve the minimum strength before exposure to freezing and thawing.

**Table 3.12 Limit for supplementary cementitious materials (ACI 201.2R)**

| Cementitious Material | Maximum allowable percent of cementitious material by mass (%) |
|---|---|
| Exposure Class - F3<br>Severity – Very severe<br>Condition – Concrete exposed to freezing and thawing conditions as well as deicing chemicals.<br>**Fly Ash** | 25 |
| Exposure Class - F3<br>Severity – Very severe<br>Condition – Concrete exposed to freezing and thawing conditions as well as deicing chemicals.<br>**Slag** | 50 |

The total amount of cementitious materials incorporated in all the mix design formulations used in this project are listed in Table 3.13 and is compared with the recommended criteria from ACI 201.2R. All mix designs satisfied the maximum content of fly ash or slag recommended by ACI 201.2R. Mix 5 has slag content slightly above the maximum at 50.2%.

**Table 3.13 Total amount of supplementary cementitious materials in mix design formulations**

| Cementitious Material | Mix 1 | Mix 2 | Mix 3 | Mix 4 | Mix 5 | Criteria |
|---|---|---|---|---|---|---|
| Fly Ash (%) | 25 | - | - | 25 | - | ≤ 25 |
| Slag (%) | - | 49.8 | - | - | 50.2 | ≤ 50 |

### 3.2.3 Chemical Admixtures

Concrete properties like air content, workability, and setting time can be altered using chemical admixtures. They can also be used to achieve specific consistency and properties during the concreting process. Some of the most commonly used admixtures are air-entraining, water-reducing, retarders, accelerators and superplasticizers. In this project water reducers, air entraining, and workability retention admixtures were used in the cement concrete mix designs.
Air-entraining admixtures are used to modify the air void system to increase the concrete workability, resistance to freezing and thawing, and to prevent segregation, reduce bleeding, and decrease water demand.

Water-reducing admixtures are beneficial during hot weather concreting to reduce the increased water demand necessary in hot weather to compensate for high concrete temperatures and evaporation. Excess water can lead to loss of concrete strength and increase permeability. Water reducers reduce the water demand but help maintain the workability of concrete. Using water-retaining and retarding admixtures results in a higher rate of slump loss.

According to the product description (product datasheet is available in Appendix) of Master Sure Z 60, the workability retention admixtures are used to provide slump retention without retardation. When used in combination with water-reducing admixtures, the increased water demand during hot weather concreting can be compensated by maintaining the workability and reducing the rate of slump loss.

### 3.2.4 Paste Content

Paste content impacts concrete mixture properties such as strength, workability, liquid retention during curing, penetrability, can prevent cracking and shrinkage from cracking. When a paste content is greater than 30 %, the concrete has a higher chance of cracking than when it is below 30 % and above 28 %, a range considered as an 'acceptable" level. Below 28 %, the concrete is at an "exceptional" paste content level in which cracking likelihood is drastically decreased.

Paste content is calculated from the ratio of volume of paste to volume of concrete, where V = volume (ft3) and PC = Paste content (%).

$$V_p = V_{cement} + V_{scm} + V_{water}$$

$$V_{concrete} = V_{cement} + V_{scm} + V_{water} + V_{ca} + V_{fine} + V_{air}$$

$$eq. 3.2\text{: } PC = \frac{V_p}{V_{concrete}}$$

The concept of Excessive Paste Content allows for the evaluation of fine and coarse aggregate binding while considering voids and aiding in workability. The Excessive Paste Content (EPC) is obtained as the sum of percentages of paste content (PC) and aggregate content (AC) and subtracting the void content (VC).

$$eq. 3.3\text{: } EPC = PC + AC - VC$$

The ratio of paste content to void content (PC/VC) can be used as an indicator of workability, with a higher PC/VC indicating higher workability. When water-reducing admixtures are not used, a lower PC/VC ratio causes a decreased workability. The PC/VC ratio is determined from a set of equations where VC = Void Content (%), UW = Unit Weight (pcf), V = Volume (ft3), SG = Specific Gravity, D = Density (pcf), PC/VC = Paste Content to Void Content ratio, and PC = Paste Content (%).

$$VC_{ca} = \frac{SG_{ca} * D_{water} - UW_{ca}}{SG_{ca} * D_{water}}$$

$$VC_{fine} = \frac{SG_{fine} * D_{water} - UW_{fine}}{SG_{fine} * D_{water}}$$

$$VC_{agg} = \frac{V_{ca}}{V_{ca} + V_{fine}} * VC_{ca} + \frac{V_{fine}}{V_{ca} + V_{fine}} * VC_{fine}$$

$$VC = \frac{V_{ca} + V_{fine}}{V_{concrete}} * VC_{agg}$$

$$eq. 3.4: PC/VC = \frac{PC}{VC}$$

Table 3.14 displays results of the paste content (PC) mixture calculations for this project.

**Table 3.14 Paste content results**

| Property | Mix 1 | Mix 2 | Mix 3 | Mix 4 | Mix 5 | Criteria |
|---|---|---|---|---|---|---|
| Paste Content (%) | 29.0 | 28.7 | 28.1 | 30.7 | 32.8 | ≤ 28[1] |
| Paste Content to Void Content (PC/VC) | 1.2 | 1.2 | 1.2 | 1.2 | 1.3 | 1.25 - 1.75[2] |
| Excessive Paste Content (%) | 11.9 | 11.5 | 10.7 | 8.5 | 10.6 | - |

Notes:
[1] A Paste Content > 30 has a "greatly increased" cracking tendency and is shown in red highlighted cells. Paste Content between 28 – 30 is acceptable for structural concrete and is shown in green highlighted cells. Paste Content between 25 – 28 is exceptional for structural concrete and < 25 is exceptional for slip form concrete.
[2] A (PC/VC) < 1.25 has decreased workability and is shown in orange highlights. A (PC/VC) between 1.25 – 1.75 is shown to have exceptional workability, and > 1.75 has issues with segregation, cracking, shrinkage, spalling, and scaling.

The paste content of mixes 1, 2, and 3 meet acceptable limits, but the paste content in mixes 4, and 5 indicate having a higher tendency to cracking.

## 3.3 Air-Void Spacing

### 3.3.1 Air Content

Field performance of a concrete design mixture, particularly those that will be subjected to de-icing chemicals and freeze-thaw cycling is dependent on properties such as air content. Table 3.15 lists criteria contained in ACI 201.2R-16 designing concrete mixtures that experience several freezing conditions for a given nominal maximum aggregate size

(NMAS) in inches. Each of the 5 concrete mixtures in this study classified as NMAS for ¾ in.  and class F3 (severe exposure condition).

**Table 3.15 Freezing, thawing, and de-icing resistance**

| Class | Exposure Condition Severity | W/CM | NMAS (in.) | AC (%) |
|---|---|---|---|---|
| F1 | Moderate:  Exposed to freezing and thawing cycles; Not exposed to accumulation of snow, ice, and de-icing chemicals; Limited exposure to water | ≤ 0.55 | 3/8 | 6 |
| F1 | Moderate | ≤ 0.55 | 1/2 | 5.5 |
| F1 | Moderate | ≤ 0.55 | 3/4 | 5 |
| F1 | Moderate | ≤ 0.55 | 1 to 1.5 | 4.5 |
| F2 | Severe:  Exposed to freezing and thawing cycles and accumulation of snow and ice; Not exposed to de-icing chemicals; Frequent exposure to water; Direct contact with soil | ≤ 0.45 | 3/8 | 7.5 |
| F2 | Severe | ≤ 0.45 | 1/2 | 7 |
| F2 | Severe | ≤ 0.45 | 3/4 to 1 | 6 |
| F2 | Severe | ≤ 0.45 | 1.5 | 5.5 |
| F3 | Very Severe:  Exposed to freezing and thawing cycles and accumulation of snow, ice, and de-icing chemicals; Frequent exposure to water | ≤ 0.40 | 3/8 | 7.5 |
| F3 | Very Severe | ≤ 0.40 | 1/2 | 7 |
| F3 | Very Severe | ≤ 0.40 | 3/4 to 1 | 6 |
| F3 | Very Severe | ≤ 0.40 | 1.5 | 5.5 |

From Table 3.16 the minimum Air Content of the 5 mixture designs for the project should be 6%. MassDOT allows a ± 1% tolerance in air content as described in the MassDOT Specifications and ACI 201.2R-16.

**Table 3.16 Air content in mixtures as designed**

| Property | Mix 1 | Mix 2 | Mix 3 | Mix 4 | Mix 5 | Criteria |
|---|---|---|---|---|---|---|
| Air Content (%) | 7.0 | 7.0 | 7.0 | 2.5 | 2.5 | > 6 |

Table 3.16 *Air Content in mixtures as designed* shows that Mixtures 1, 2, and 3 satisfy the air content threshold for the F3 class of exposure due to very serve freezing, thawing, and de-icing resistance. Mixtures 4 and 5 do not satisfy the threshold for F1 class of moderate exposure to freezing, thawing, and de-icing resistance. AASHTO T 152 *Air Content of Freshly Mixed Concrete by the Pressure Method* was performed and results are reported in Table 4.5 Fresh Concrete Properties Test Results and again can be compared to Table 3.15 *Freezing, Thawing, and De-icing resistance* following provisions in ACI 201.2R and the MassDOT Specifications.

### 3.3.2 Air Entraining Chemical Admixtures

Mix designs with air entraining admixtures must comply with ASTM C260 *Standard Specification for Air-Entraining Admixtures for Concrete.* Properties of air entraining admixtures are described in ACI Education Bulletin E4-12 *Chemical Admixtures for Concrete.* Air entraining admixtures are particularly beneficial for workability in mixtures containing supplementary cementitious materials, they moderate bleeding and segregation, and are intended to protect from scaling due to freezing, thawing, and de-icing resistance. Air entraining chemical admixtures also support against sulfate reaction and alkali-reactive environments. Table 3.3 *Chemical Admixtures* list the Master Air AE 200 air entraining as the admixture labeled AD1, which appears in Table 3.4 as used in mix design formulations for this project. This air entraining admixture was used in Mixes 1, 2, and 3. Mixes 4 and 5 were intentionally designed to not include such air entraining chemical admixtures.

# 4.0 Field and Laboratory Testing

This chapter includes results of all the tests conducted on aggregates, fresh concrete, and hardened concrete. Tests were conducted at various organizations involved in the study. The tests described herein were conducted at MassDOT, UMass Amherst, or Wiss Janney Elstner Associates, Inc. (WJE).

## 4.1 Aggregate Tests

The test methods conducted on aggregates properties are described in Table 4.1 and the results are compared with the criteria in Table 4.2 and 4.3.

**Table 4.1 Aggregates test methods**

| Test Method | Description |
|---|---|
| T 19 | Bulk Density ("Unit Weight") and Voids in Aggregate |
| T 27 | Sieve Analysis of Fine and Coarse Aggregate |
| T 84 | Specific Gravity and Absorption of Fine Aggregate |
| T 85 | Specific Gravity and Absorption of Coarse Aggregate |

**Table 4.2 Fine aggregates test results**

| Test Method | Property | Result | Criteria |
|---|---|---|---|
| T 19 | Unit Weight (lb./ ft3) | 108.7 | - |
| T 19 | Aggregate Void Content (%) | 34.4 | - |
| T 84 | Bulk Specific Gravity (Dry) | 2.66 | - |
| T 27 | Percentage by Mass Passing (%) 3/8 in. | 100.0 | 100 |
| T 27 | Percentage by Mass Passing (%) No. 4 | 100.0 | 95-100 |
| T 27 | Percentage by Mass Passing (%) No. 8 | 92.3 | 80-100 |
| T 27 | Percentage by Mass Passing (%) No. 16 | 77.7 | 50-85 |
| T 27 | Percentage by Mass Passing (%) No. 30 | 47.1 | 25-60 |
| T 27 | Percentage by Mass Passing (%) No. 50 | 16.0 | 10-30 |
| T 27 | Percentage by Mass Passing (%) No. 100 | 5.1 | 2-10 |
| T 27 | Percentage by Mass Passing (%) No. 200 | 2.5 | 0-3 |
| T 27 | Fineness Modulus (FM) | 2.62 | 2.3-3.1 |

**Table 4.3 Coarse aggregates test results**

| Test Method | Property | Result | Criteria |
|---|---|---|---|
| T 19 | Unit Weight (lb/ ft3) | 108.4 | - |
| T 19 | Aggregate Void Content (%) | 40.0 | - |
| T 84<br>T 85 | Bulk Specific Gravity (Dry) | 2.90 | - |
| T 27 | Percentage by Mass Passing (%)<br>1 in. | 100.0 | 100 |
| T 27 | Percentage by Mass Passing (%)<br>3/4 in. | 98.1 | 90-100 |
| T 27 | Percentage by Mass Passing (%)<br>1/2 in. | 66.8 | - |
| T 27 | Percentage by Mass Passing (%)<br>3/8 in. | 35.3 | 20-55 |
| T 27 | Percentage by Mass Passing (%)<br>No. 4 | 6.5 | 0-10 |
| T 27 | Percentage by Mass Passing (%)<br>No. 8 | 2.1 | 0-50 |
| T 27 | Percentage by Mass Passing (%)<br>No. 16 | 1.8 | - |
| T 27 | Fineness Modulus (FM) | 6.52 | - |

## 4.2 Fresh Concrete Tests

The properties of fresh concrete are identified by conducting the test methods described in Table 4.4. The results of the test methods for all mix design formulations are compared with criteria in Table 4.5

**Table 4.4 Fresh concrete properties test methods**

| 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 129 | Vibrating Kelly Ball (Vkelly) Penetration in Fresh Portland Cement Concrete |
| T 318 | Water Content of Freshly Mixed Concrete Using Microwave Oven Drying |

**Table 4.5 Fresh concrete properties test results**

| Test Method | Property | Mix 1 | Mix 2 | Mix 3 | Mix 4 | Mix 5 | Criteria |
|---|---|---|---|---|---|---|---|
| T 119 | Slump (in) | 6 | 2 | 5 | 4.5 | 3.25 | 3.5-6.5 |
| T 121 | Unit Weight (lb/ft$^3$) | 147.2 | 146.3 | 147.4 | 147.9 | 154.2 | - |
| T 152 | Air Content (%) | 6.2 | 5 | 6.2 | 3.1 | 1.5 | 7.0 ± 1.5 |
| T 309 | Concrete Temperature (F) | 82 | 88 | 88 | 86 | 90 | 60-90 |
| TP 129 | Average Vkelly Slump (in) | 2.83 | 0.50 | 2.17 | 1.50 | 1.17 | - |
| TP 129 | Average Vkelly Index (in/√s) | 0.52 | 0.62 | 0.69 | 0.96 | 0.39 | - |
| T 318 | Total Measured Water Content (lb/yd$^3$) | 513 | 741 | 314 | 222 | 694 | - |

### 4.2.1 VKelly Penetration in Fresh Portland Cement Concrete

Following AASHTO Test Specification TP 129-18 (Vibrating Kelly Ball Penetration in Fresh Portland Cement Concrete), three Vibrating Kelly Ball tests were performed on each mix to determine the concrete slump. The resulting VKelly slump is compared to both design slump and cone measured slump. Given that VKelly test is designated for mixes with low slump, the mixes tested in this study are not ideal for the test, as per the standard. In some cases, this resulted in the Vibrating Ball reaching the bottom of the rubber basin before the standard 36 second measuring period had elapsed.

The initial static reading ($R_i$) is taken by moving the Kelly Ball until it contacts the surface of the concrete without releasing its weight. The final static reading ($R_s$) is taken by statically releasing the Kelly Ball on the surface of the concrete and measuring the distance it sinks into the mix.

**Figure 4.1 Project worker performing Vibrating Kelly Ball test on the concrete**

**Table 4.6 Static Kelly Ball test readings**

| Mix Design | $R_i$, Initial Static Reading (in) | $R_s$, Final Static Reading (in) | $D_s$, Static Load Depth of Penetration (in) | Average Slump Equivalent (in) | Cone Measured Slump (in) |
|---|---|---|---|---|---|
| 1 | 3 3/4 | 5 3/4 | 2 | 2.833 | 6 |
| 1 | 4 1/4 | 5 1/4 | 1 | 2.833 | 6 |
| 1 | 3 1/2 | 4 3/4 | 1 1/4 | 2.833 | 6 |
| 2 | 3 1/4 | 3 3/4 | 1/2 | 0.500 | 2 |
| 2 | 3 1/4 | 3 1/2 | 1/4 | 0.500 | 2 |
| 2 | 3 | 3 | 0 | 0.500 | 2 |
| 3 | 2 3/4 | 4 1/4 | 1 1/2 | 2.167 | 4.25 |
| 3 | 2 3/4 | 4 | 1 1/4 | 2.167 | 4.25 |
| 3 | 3 1/4 | 3 3/4 | 1/2 | 2.167 | 4.25 |
| 4 | 3 1/2 | 4 1/4 | 3/4 | 1.500 | 4.5 |
| 4 | 3 | 4 | 1 | 1.500 | 4.5 |
| 4 | 3 | 3 1/2 | 1/2 | 1.500 | 4.5 |
| 5 | 3 1/4 | 4 | 3/4 | 1.167 | 3.5 |
| 5 | 3 | 3 3/4 | 3/4 | 1.167 | 3.5 |
| 5 | 3 1/4 | 3 1/2 | 1/4 | 1.167 | 3.5 |

Slump equivalent is calculated by multiplying the average depth of penetration under static load ($R_s$) determined from three test repetitions by two. The test procedure then involves conducting a dynamic test by vibrating the Kelly Ball into the concrete mix using a standard concrete vibrator attached to the Kelly Ball for 36 seconds and noting the depth for each 6 seconds.

A plot depicting vibration duration as a function of penetration depth is constructed and the slope of the best fit line through these data points is used to determine the VKelly Index. The VKelly index is a tool to determine how fluid a mixture becomes due to vibration[1]. This allows comparison of concrete mixtures which have similar slump values. A VKelly index > 0.6 (in/√s) is difficult to consolidate, and < 1.1 (in/√s) would display edge slump. Penetration Depth is the difference between the Final Static Reading (Rs) and the Dynamic Reading for each instance. Figure 4.2 shows the plot for Mixture 1, plots of the other mixtures can be found in Appendix F.

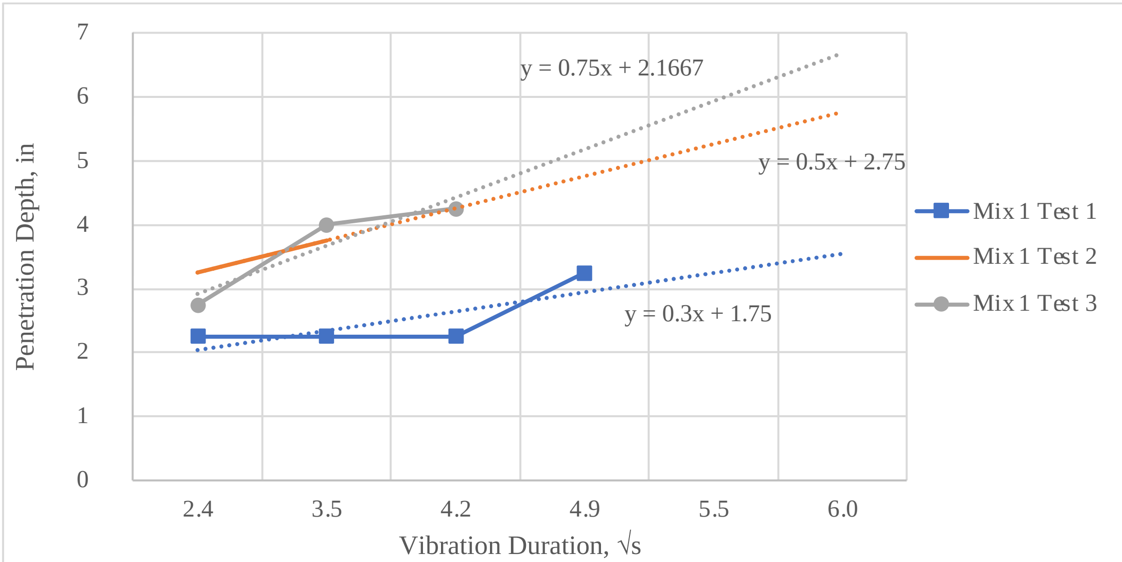



**Figure 4.2 Vibrating Kelly mixture 1 tests 1,2, and 3**

Table 4.7 shows the VKelly Test summary of results for each of the mixtures. The individual VKelly indexes are averaged to determine the overall mixture VKelly Index.

**Table 4.7 Vibrating Kelly Ball in fresh Portland cement concrete test results**

| Mix Design | VKelly Index (in/√s) | Average VKelly Index (in/√s) | Average VKelly Slump (in) | Design Slump (in) | Cone Measured Slump (in) |
|---|---|---|---|---|---|
| 1 | 0.5 | 0.52 | 2.83 | 5 | 6 |
| 1 | 0.75 | 0.52 | 2.83 | 5 | 6 |
| 1 | 0.3 | 0.52 | 2.83 | 5 | 6 |
| 2 | 0.775 | 0.62 | 0.50 | 5 | 2 |
| 2 | 0.5357 | 0.62 | 0.50 | 5 | 2 |
| 2 | 0.5357 | 0.62 | 0.50 | 5 | 2 |
| 3 | 0.875 | 0.69 | 2.17 | 5 | 4.25 |
| 3 | 0.65 | 0.69 | 2.17 | 5 | 4.25 |
| 3 | 0.55 | 0.69 | 2.17 | 5 | 4.25 |
| 4 | 0.5 | 0.96 | 1.50 | 5 | 4.5 |
| 4 | 1.25 | 0.96 | 1.50 | 5 | 4.5 |
| 4 | 1.125 | 0.96 | 1.50 | 5 | 4.5 |
| 5 | 0.479 | 0.39 | 1.17 | 6 | 3.5 |
| 5 | 0.3429 | 0.39 | 1.17 | 6 | 3.5 |
| 5 | 0.25 | 0.39 | 1.17 | 6 | 3.5 |

## 4.3 Hardened Concrete Testing

The hardened concrete properties test methods are described in Table 4.8 and Table 4.9 consists of results of hardened concrete properties tests for all mix design formulations. The standard ASTM C672 was withdrawn in 2021.However, the test was conducted in this study to establish a comparison with BNQ NQ 2621-900.

**Table 4.8 Hardened concrete properties tests**

| 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 |
| C672 | Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals |
| BNQ NQ 2621-900 | Determination of the Scaling Resistance of Concrete Surfaces Exposed to Freezing and Thawing Cycles in the Presence of Deicing Chemicals |
| TP 119 | Electrical Resistivity of a Concrete Cylinder Tested in a Uniaxial Resistance Test |
| TP 135 | Determining the Total Pore Volume in Hardened Concrete Using Vacuum Saturation |
| TP 136 | Determining the Degree of Saturation of Hydraulic-Cement Concrete |

**Table 4.9 Hardened concrete properties tests results**

| Test Method | Property | Mix 1 | Mix 2 | Mix 3 | Mix 4 | Mix 5 | Criteria |
|---|---|---|---|---|---|---|---|
| T 22 | Compressive Strength (psi) 7 days | - | 3080 | 3650 | 2320 | - | ≥ 2800 |
| T 22 | Compressive Strength (psi) 28 days | 3840 | 4890 | 4640 | 3270 | 5380 | ≥ 4000 |
| T 22 | Compressive Strength (psi) 91 days | 4910 | 5750 | 5130 | 4250 | 6330 | ≥ 4000 |
| T 358 | Resistivity (kohm-cm) 7 days | 5.3 | 7.5 | 6.3 | 4.4 | 8.5 | - |
| T 358 | Resistivity (kohm-cm) 28 days | 11.4 | 17.2 | 8.9 | 6.6 | 30.8 | ≥ 21.0 |
| T 358 | Resistivity (kohm-cm) 91 days | 21.5 | 32.2 | 10.9 | 13.5 | 34.4 | ≥ 21.0 |
| C 672 | Scaling Resistance: Standard Moist Cure (Rating) | 2.5 | 5 | 1.5 | 5 | 5 | ≤ 2.0 |
| NQ 2621-900[2] | Scaling Resistance: Curing using Saturated Cover (kg/m2) | 0.03 | 0.04 | 0.01 | 1.21 | 0.19 | < 0.5 |
| NQ 2621-900[2] | Scaling Resistance: Curing using Sealing and Curing Compound (kg/m2) | 0.46 | 1.33 | 0.45 | 3.51 | 0.98 | < 0.5 |
| NQ 2621-900[2] | Scaling Resistance: Curing using Colloidal Silica Sealer | - | - | 0.25 | - | - | < 0.5 |
| NQ 2621-900[2] | Scaling Resistance: No Curing (kg/m2) | 0.22 | 0.88 | 0.08 | 4.56 | 2.22 | < 0.5 |
| TP 119[3,4] | Uniaxial Resistivity (kohm-cm) 7 days | 3.07 | 4.06 | 3.54 | 2.24 | 4.66 | - |
| TP 119[3,4] | Uniaxial Resistivity (kohm-cm) 28 days | 5.32 | 9.34 | 4.45 | 3.61 | 11.6 | - |
| TP 119[3,4] | Uniaxial Resistivity (kohm-cm) 91 days | 12.1 | 17.6 | 5.98 | 7.33 | 19.4 | - |
| TP 135 [4] | Volume of Permeable Pore Volume (%) | 13.8 | 14.38 | 14.33 | 16.48 | 14.07 | |
| TP 136 [4] | Degree of Saturation (%) | 0.04 | 0.05 | 0.04 | 0.03 | 0.06 | |

Notes:

[1] Orange cells indicate values exceeding the given criteria

[2] Various curing methods are elaborated in section 4.3. The number of freeze thaw cycles per the test is 56 cycles but extended until 112 cycles. The results in this table are for 56 freeze-thaw cycles.

[3] Uniaxial resistivity is tested for specimens which were subjected to ambient temperature for several months and conditioned before the test. The results of these additional tests are elaborated in section 4.3.3

[4] Test procedure and results are elaborated in further sections

### 4.3.1 Determining the Total Pore Volume in Hardened Concrete Using Vacuum Saturation

The Total Pore Volume (AASHTO TP 135-20) test procedure was followed to determine the Volume of Permeable Pore Volume (%) and the Mass Increase Due to Saturation (%). The vacuum desiccator used in this test allows for complete saturation which includes air entrained voids, unlike ASTM C642. The inclusion of air entrained voids means total pore volume can be measured.

Following this, the vacuum pressure is released, and the specimens remain submerged under The results of the Total Pore Volume test are summarized in Table 4.10.

The volume of permeable pore volume can be calculated from equation 4.1, where A = the mass of oven-dried sample in air (g), B = the mass of saturated, surface dry sample in air after vacuum (g), and C = apparent mass of sample in water after vacuum (g)

$$Eq. 4.1: Volume\ of\ permeable\ pore\ volume = \frac{B - A}{B - C} * 100$$

The percentage of mass increase due to saturation can be calculated from the following equation, where C = the apparent mass of sample in water after vacuum (g), and A = mass of oven-dried sample in air (g).

$$Eq. 4.2: Mass\ increase\ due\ to\ saturation, D = \frac{C}{A} * 100$$

**Table 4.10 Total pore volume test results**

| Specimen | Volume of permeable pore volume (%) | Mass increase due to saturation (%) |
|---|---|---|
| Mix 1 | 13.80 | 55.25 |
| Mix 2 | 14.38 | 55.54 |
| Mix 3 | 14.33 | 54.26 |
| Mix 4 | 16.48 | 55.53 |
| Mix 5 | 14.07 | 57.59 |

Due to the same curing, mix design, and placement environment, it can be presumed that the volume of permeable pore volume represents the same properties as the in-situ sidewalk panels. The volume of permeable pore volume is the percentage of voids in the cylinder specimen. Total pore volume can be used in calculations of durability and transport.

### 4.3.2 Determining the Degree of Saturation of Hydraulic-Cement Concrete

The Degree of Saturation of Hydraulic-Cement Concrete (AASHTO TP 136-20) test procedure was followed to determine the Degree of Saturation. The Degree of Saturation test is a continuation of the total pore volume (AASHTO TP 135-20) test. The mass increase due to saturation from TP 135 can be used in calculations of the degree of saturation.

The variance is in the conditioning of the cylinders prior to testing. The cylinders are preconditioned at a relative humidity of 37% at 23 ± 2 °C for at least 24 hours, this is presented as the original conditioned mass. Otherwise, the test procedure is the same as TP 135-20.

The results of the degree of saturation test are summarized in Table 4.111. The degree of saturation can be calculated from equation 4.3, where E = the conditioned mass of the specimen (g), A = the mass of oven-dried sample in air (g), D = mass increase due to saturation (%), and B = mass of saturated, surface-dry sample in air after vacuum

$$Eq. 4.3: DOS = \frac{E - A}{A * (D - 1)} * 100\%$$

**Table 4.11: Degree of saturation test results**

| Specimen | Degree of Saturation (%) | Mass Increase due to saturation (%) |
|---|---|---|
| Mix 1 | 0.04 | 56.21 |
| Mix 2 | 0.05 | 56.03 |
| Mix 3 | 0.04 | 56.57 |
| Mix 4 | 0.03 | 56.97 |
| Mix 5 | 0.06 | 58.47 |

Due to the same curing, mix design, and placement environment, it can be presumed that the degree of saturation represents the same properties as the in-situ sidewalk panels. Degree of saturation can be an indicator of scaling in cases of ponding or saturation of a concrete specimen.

### 4.3.3 Electrical Resistivity of a Concrete Cylinder Tested in a Uniaxial Resistance Test

The electrical resistivity of concrete can act as an indicator for the resistance of concrete against the penetration of chloride ions. From previous studies, it has been determined that various factors influence concrete's electrical resistivity, such as temperature, sample conditioning, geometry, degree of saturation, age of the sample, and pore structure of concrete.

AASHTO TP 119-15 (2019) *Electrical Resistivity of a Concrete Cylinder Tested in a Uniaxial Resistance Test* has been followed for this study. As per the standard, the chloride ion penetrability classification can be determined for any concrete sample based on its uniaxial resistivity.

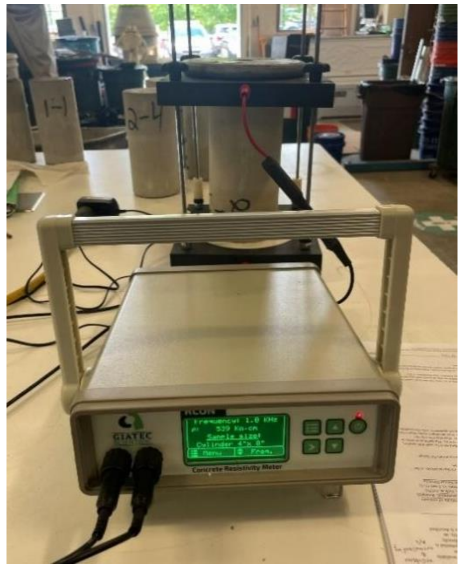



**Figure 4.3 Concrete cylinder tested in concrete resistivity meter for uniaxial resistivity**

The AASHTO TP 119 test was performed for two sets of samples at the MassDOT research facility and UMass Amherst Boyle Structural Engineering Lab. At MassDOT, the concrete cylinders were subjected to a standard curing method and immediately put in an environmental chamber for conditioning until the day of testing. The test was performed after conditioning the samples for 7, 28, and 91 days. The tests done at MassDOT have followed the standard precisely.

The tests performed at UMass Amherst lab were conducted by modifying the standard curing and sample conditioning procedures stipulated in AASHTO TP 119. The cylinders of 8-in length and 4-in diameter at UMass were prepared during the placement of concrete sidewalks and moisture cured for 28 days. These cylinders were subjected to temperature and humidity conditions present in the laboratory. They were allowed to mature before testing for electrical resistivity after nine months. Subsequently, these concrete cylinders were conditioned for 7 and 28 days in saturated lime solution before testing again, which was done to identify if the age of concrete and moisture content in concrete affect the concrete resistivity. The test on samples exposed to room temperature for nearly nine months might better represent field conditions as the concrete slabs on the field are not conditioned after curing. The concrete cylinders were in saturated surface dry (SSD) condition before testing in a concrete resistivity meter (Giatec RCON). Figure 4.7 shows a cylinder in the electrical resistivity test frame.

**Table 4.12 Uniaxial resistivity and chloride ion penetrability classification of cylinders test at MassDOT**

| Conditioning of specimen | Mix 1 | Mix 2 | Mix 3 | Mix 4 | Mix 5 |
|---|---|---|---|---|---|
| 7 Days Resistivity (kohm.cm) | 3.07 | 4.06 | 3.54 | 2.24 | 4.66 |
| 7 Days Classification | High | High | High | High | High |
| 28 Days Resistivity (kohm.cm) | 5.32 | 9.34 | 4.45 | 3.61 | 11.60 |
| 28 Days Classification | Moderate | Moderate | High | High | Low |
| 91 Days Resistivity (kohm.cm) | 12.10 | 17.60 | 5.98 | 7.33 | 19.40 |
| 91 Days Classification | Low | Low | Moderate | Moderate | Low |

**Table 4.13 Uniaxial resistivity and chloride ion penetrability classification of cylinders test at UMass Amherst**

| Conditioning of specimen | Mix 1 | Mix 2 | Mix 3 | Mix 4 | Mix 5 |
|---|---|---|---|---|---|
| No Conditioning Resistivity (kohm.cm) | 1480.0 | 516.0 | 1100.0 | 1916.7 | 212.0 |
| No Conditioning Classification | Neligible | Neligible | Neligible | Neligible | Neligible |
| 7 Days Resistivity (kohm.cm) | 12.5 | 26.5 | 6.5 | 6.5 | 26.6 |
| 7 Days Classification | Low | Very Low | Moderate | Moderate | Very Low |
| 28 Days Resistivity (kohm.cm) | 12.5 | 27.2 | 6.1 | 7.2 | 29.1 |
| 28 Days Classification | Low | Very Low | Moderate | Moderate | Very Low |

The electrical resistivity (Kohm-cm) of each sample and its chloride ion penetrability classification tested at MassDOT are shown in Table 4.12. From the table, it can be concluded that the resistivity of concrete increases as the concrete matures. Mix 3, which does not contain supplementary cementitious materials, has the lowest resistivity after 91 days. Table 4.13 consists of electrical resistivity and chloride ion penetrability of UMass Amherst cylinders. The cylinders, which are conditioned for 7 and 28 days, represent field conditions; when the sidewalks moisture content is changed after being subjected to rain or snow. The resistivity of the cylinders is high when no conditioning is done on the cylinders.

After conditioning, the resistivity is close to MassDOT 91 days conditioning cylinders except Mix 2 and 5, which consist of 50% slag. Mix 2 and 5 after 28 days of conditioning have a 54.55% and 50% increase in resistivity compared to MassDOT cylinders with 91 days of conditioning. It can be concluded from the results that as concrete matures, the resistivity of concrete increases. Therefore, the chloride ion penetrability decreases.

## 4.4 Scaling Resistance Tests

To determine scaling resistance of the concrete mixes in this project, 10 samples were tested per ASTM C762 at the Massachusetts Department of Transportation (MassDOT) Research Laboratory and 32 samples were tested following Quebec Standard BNQ NQ 2621-900 at University of Massachusetts Amherst (UMass Amherst). The information on samples and test protocols are discussed in length in sections 4.4.1 and 4.4.2. The samples tested at MassDOT were subjected to standard curing as per ASTM C672 *(2)*. At UMass, the 32 samples were cast during the placement of sidewalks at Brack Laboratory, as shown in Figure 3.1. The samples were subjected to the same curing methods and environmental conditions as their corresponding sidewalk panels. The curing methods and environmental conditions at the site are discussed in section 5.2. The scaling test specimens were cured immediately next to the corresponding sidewalks panels and were subjected to identical environmental conditions.

A critical difference in the two standards is in the treatment of how scaling is measured. Another difference is that for BNQ NQ 2621-900, the samples are pre-saturated for 7 days with 3% NaCl solution prior to freeze-thaw cycles, which is not followed by the ASTM C672 standard. In ASTM C672, a visual rating of the samples subjected to freeze-thaw cycles is made at 5, 10, 15, 25, and 50 cycles *(2)*. For BNQ NQ 2621-900, scaled-off mass from the sample is collected at 7, 21, 35, and 56 cycles. To enable direct comparison between the two test protocols, a visual rating was also taken for the BNQ test series, although the standard does not require it.


**Figure 4.4: Placement of 6 in. x 12 in. x 3in. scaling test specimens**

### 4.4.1 ASTM C672 (MassDOT Laboratory)

The ASTM C672 test was performed at Massachusetts Department of Transportation (MassDOT) Research Laboratory. The specimen size is 6 in. by 12 in. by 3 in. deep, subjected to 50 freeze-thaw cycles at -18°C for 16 to 18 hours, then 23°C for 6 to 8 hours with a solution of 4% calcium chloride ponded on the surface of the specimens for the duration. Specimens are visually rated on cycles 5, 10, 15, 25, and 50 on a scale of 0 to 5 as defined in the test standard. Once a specimen reached visual rating of 5, the rating is reported as 5 through the reminder.

**Table 4.14 ASTM C672 Standard test method for scaling resistance of concrete surfaces exposed to deicing chemicals**

| Mix Design No[1] | Dates | Rating | Mix Design No[1] | Dates | Rating |
|---|---|---|---|---|---|
| 1a | 08/30/2021 | 0 | 1b | 08/30/2021 | 1 |
| 1a | 09/07/2021 | 1 | 1b | 09/07/2021 | 1 |
| 1a | 09/12/2021 | 2 | 1b | 09/12/2021 | 1 |
| 1a | 09/29/2021 | 3 | 1b | 09/29/2021 | 2 |
| 1a | 11/03/2021 | 3 | 1b | 11/03/2021 | 2 |
| 2a | 08/30/2021 | 1 | 2b | 08/30/2021 | 1 |
| 2a | 09/07/2021 | 1 | 2b | 09/07/2021 | 1 |
| 2a | 09/12/2021 | 1 | 2b | 09/12/2021 | 2 |
| 2a | 09/29/2021 | 3 | 2b | 09/29/2021 | 4 |
| 2a | 11/03/2021 | 5 | 2b | 11/03/2021 | 5 |
| 3a | 08/30/2021 | 0 | 3b | 08/30/2021 | 1 |
| 3a | 09/07/2021 | 0 | 3b | 09/07/2021 | 1 |
| 3a | 09/12/2021 | 0 | 3b | 09/12/2021 | 1 |
| 3a | 09/29/2021 | 0 | 3b | 09/29/2021 | 1 |
| 3a | 11/03/2021 | 1 | 3b | 11/03/2021 | 2 |
| 4a | 08/30/2021 | 4 | 4b | 08/30/2021 | 5 |
| 4a | 09/07/2021 | 5 | 4b | 09/07/2021 | 5 |
| 4a | 09/12/2021 | 5 | 4b | 09/12/2021 | 5 |
| 4a | 09/29/2021 | 5 | 4b | 09/29/2021 | 5 |
| 4a | 11/03/2021 | 5 | 4b | 11/03/2021 | 5 |
| 5a | 08/30/2021 | 2 | 5b | 08/30/2021 | 2 |
| 5a | 09/07/2021 | 5 | 5b | 09/07/2021 | 5 |
| 5a | 09/12/2021 | 5 | 5b | 09/12/2021 | 5 |
| 5a | 09/29/2021 | 5 | 5b | 09/29/2021 | 5 |
| 5a | 11/03/2021 | 5 | 5b | 11/03/2021 | 5 |

Note:

[1] Two samples (a and b) are tested for each mix design.

### 4.4.2 BNQ NQ 2621-900 (UMass Laboratory)

For *Quebec Standard BNQ NQ 2621-900* test, the sample size is 6 in. by 12 in. by 3 in. deep. Dikes were constructed using plexiglass to pond brine solution during freeze-thaw cycles. Plexiglass strips were attached and sealed to the sides of the concrete samples with a clearance of 1 in. Two layers of strips were attached, interfacing at corners, and edges were sealed along the perimeter and at intersections, as can be seen in Figure 4.9.


**Figure 4.5 Plexiglass dikes installed on a specimen**

A 3% NaCl solution was created for the test by mixing 300 g of NaCl in 10 liters of water until the salt dissolves. The specimens were placed during the placement of sidewalks and were subjected to same curing methods as the sidewalks. After curing, the specimens were kept at room temperature until dikes were built in November. Once testing commenced in November 2021, specimens were pre-saturated for seven days with brine solution. The depth of the brine solution on the specimen surface is 5 ± 3 mm. During freeze-thaw cycles, the top of plexiglass dams were covered with plastic wrapping to limit the evaporation of brine solution as shown in Figure 4.10.

**Figure 4.6 Specimens in chest freezer**

The concrete specimens were placed in a freezer chest at -18 ± 3 ºC for 16 ± 1 hours and placed in a room at laboratory temperature to thaw for 8 ± 1 hours in the Boyle Structural Engineering Laboratory at UMass. When campus closed due to inclement weather, specimens were left in the freezer at -18 ± 3 ºC until a thaw cycle could continue. Therefore, when a pause was taken, one cycle was recorded as one freezing period of more than 16 hours and regular thawing period of 8 hours. Throughout the standard 56 freeze-thaw cycles test required, only four pauses were taken.

For BNQ NQ 2621-900, a visual rating is not required. However, a visual rating is taken after collecting the mass of scaled material after 7, 21, 35, and 56 cycles. Visual rating is done using ASTM C672 visual rating scale. In addition, pictures of the surface of the specimen are regularly taken to observe the deterioration.

Table 4.15 displays the ASTM C672 visual rating scale from *ASTM C672* (2003).

**Table 4.15: ASTM C672 visual rating**

| Condition of Surface | Visual Rating |
|---|---|
| No Scaling | 0 |
| Very slight scaling<br>(3 mm or 1/8 in. depth max, no coarse aggregate visible) | 1 |
| Slight to Moderate Scaling | 2 |
| Moderate Scaling<br>(some coarse aggregate visible) | 3 |
| Moderate to Severe Scaling | 4 |
| Severe Scaling<br>(coarse aggregate visible across surface) | 5 |

The BNQ NQ 2621-900 standard utilizes scaled-off mass as failure limit state. If the scaled off mass is above 0.5 kg/m$^2$, the specimen has failed. With a specimen surface area of 0.046 m$^2$, this corresponds to a mass of 23g. Because 50% of specimens have not failed by cycle 56, the test was extended until cycle 112.

To track the progression of scaling over time, the scaled-off material from the surface of the concrete samples was periodically weighed at cycles 7, 21, 35, 56, 77, 100, and 112. Then, the surface was rinsed with the brine solution into a No. 200 sieve, and the collected materials were placed into tin foil pans. After rinsing, the brine solution was replenished, and specimens were placed back in the chest freezer for continued cycling. The pans containing scaled-off material were placed in a drying oven (Figure 4.11) at a temperature of 105 ± 5 $^{o}$C for several days until a constant mass was reached.

**Figure 4.7 Tin foil pans containing scaled-off mass in oven for drying**

### 4.4.3 Cumulative Scaled off Mass Results

The cumulative mass collected over 56 cycles for all specimens is listed in Table 4.16. The passing limit for the BNQ NQ 2621-900 test 0.5 kg/m2. Mix 1 and 3 demonstrated superior resistance to salt scaling compared to other mix designs. A comparison of curing methods for Mix 1 shows that the collected scaled-off material of samples cured with saturated cover (MCS) was 86.6% and 93.57% less than that collected form compound curing (CC) and no curing (NC) specimens, respectively. In Mix 1, no curing specimens performed better than compound curing specimens by 70%. The colloidal silica sealer (CSS) was only used in Mix 3 and performed better than similar Mix 3 specimens where compound curing (CC) was sued by 56%. All Mix 4 specimens failed by cycle 21, therefore the curing method had minimal impact for this mix. For Mix 2 and 5, only samples which were moisture cured (MCS) passed the passing limit of 0.5 kg/m2. From the samples of Mix 2 which were moisture cured (MCS) 4g of scaled-off material was collected compared to the specimens which were compound cured (CC) and no curing (NC), where the collected scaled-off mass was 122.5g and 80.9g. For Mix 5, moisture cured specimens (MCS) generated less material than compound cured (CC) and no cured (NC) specimens by 80.9% and 91.56%, respectively.

**Table 4.16 Total scaled off mass after 56 freeze thaw cycles**

| Mix No. | Curing Method | No. of Specimen | Specimen Name | Cumulative Mass (g) |
|---|---|---|---|---|
| 1 | Compound Curing | 1 | 1-CC-01 | 26.3 |
| 1 | Compound Curing | 2 | 1-CC-02 | 15.7 |
| 2 | Compound Curing | 1 | 2-CC-01 | 66.8 |
| 2 | Compound Curing | 2 | 2-CC-02 | 55.7 |
| 3 | Compound Curing | 1 | 3-CC-01 | 14.5 |
| 3 | Compound Curing | 2 | 3-CC-02 | 27.1 |
| 3 | Colloidal Silica Sealer | 1 | 3-CSS-01 | 15.1 |
| 3 | Colloidal Silica Sealer | 2 | 3-CSS-02 | 8.3 |
| 4 | Compound Curing | 1 | 1-CC-01 | 116.1 |
| 4 | Compound Curing | 2 | 1-CC-02 | 206.4 |
| 5 | Compound Curing | 1 | 2-CC-01 | 31.5 |
| 5 | Compound Curing | 2 | 2-CC-02 | 59.0 |
| 1 | Moisture Curing | 1 | 1-MCS-01 | 0.4 |
| 1 | Moisture Curing | 2 | 1-MCS-02 | 2.3 |
| 2 | Moisture Curing | 1 | 2-MCS-01 | 3.4 |
| 2 | Moisture Curing | 2 | 2-MCS-02 | 0.6 |
| 3 | Moisture Curing | 1 | 3-MCS-01 | 0.1 |
| 3 | Moisture Curing | 2 | 3-MCS-02 | 0.5 |
| 4 | Moisture Curing | 1 | 4-MCS-01 | 43.0 |
| 4 | Moisture Curing | 2 | 4-MCS-02 | 68.6 |
| 5 | Moisture Curing | 1 | 5-MCS-01 | 14.0 |
| 5 | Moisture Curing | 2 | 5-MCS-02 | 3.2 |
| 1 | No Curing | 1 | 1-NC-01 | 12.4 |
| 1 | No Curing | 2 | 1-NC-02 | 7.8 |
| 2 | No Curing | 1 | 2-NC-02 | 29.3 |
| 2 | No Curing | 2 | 2-NC-02 | 51.6 |
| 3 | No Curing | 1 | 3-NC-01 | 1.6 |
| 3 | No Curing | 2 | 3-NC-02 | 5.3 |
| 4 | No Curing | 1 | 4-NC-01 | 197.8 |
| 4 | No Curing | 2 | 4-NC-02 | 222.0 |
| 5 | No Curing | 1 | 5-NC-01 | 92.2 |
| 5 | No Curing | 2 | 5-NC-02 | 111.7 |

Note:

[1] All the cells highlighted in orange have passed the 0.5 kg/m$^2$ limit of BNQ NQ 2621-900

Table 4.17 shows cumulative mass loss at cycle 112. On a few occasions, a specimen was removed from the test program because either (1) the brine solution permeated through the concrete before measurement cycle was reached, or (2) the brine solution leaked from the dike edges because of concrete spalling. While dike repair was attempted in every instance leaking was detected, damage was frequently too severe to avoid leakage. Notably, these issues appeared well after the specified 56 cycles. For example, 2-MCS-01 and 5-MCS-02 were pulled from the test after 77 cycles. After 35 cycles, the salt solution on the 5-MCS-02 surface started leaking, which could explain why less scaled-off material was collected when compared to its companion 5-MCS-01. The salt solution completely soaked through the concrete within the first few hours of a freeze-thaw cycle.

All specimens subjected to compound curing (CC) failed the passing limit of BNQ NQ 2621-900. 18% of the specimens survived until 112 cycles with scaled off mass less than 23g at cycle 112. Curing with a saturated cover (MCS) had demonstrated superior resistance to salt scaling compared to other curing methods. Mix 1 specimens which were moisture cured (MCS) gathered 89.21% and 83.2% less scaled-off material than those cured with curing compound (CC) and no curing (NC) respectively. Additionally, Mix 1 specimens subjected to no curing (NC) outperformed those cured with a chemical compound (CC) by 43.65%. For Mix 3, all the specimens subjected to no curing (NC) and moisture curing (MCS) collected a scaled-off mass less than 10g. In Mix 3, using a colloidal silica sealer (CSS) proved to be more resistant by passing the test and collecting 38.8% less scaled material than specimens cured with chemical compound (CC). There is no change in Mix 2 and 4 samples after 56 cycles; only 2-MCS passed the 0.5 $kg/m^2$ limit.

**Table 4.17 Total scaled-off mass after 112 cycles**

| Mix No. | Curing Method | No. of Specimen | Specimen Name | Cumulative Mass (g) |
|---|---|---|---|---|
| 1 | Compound Curing | 1 | 1-CC-01 | 30.3 |
| 1 | Compound Curing | 2 | 1-CC-02 | 24.4 |
| 2 | Compound Curing | 1 | 2-CC-01 | 66.8 |
| 2 | Compound Curing | 2 | 2-CC-02 | 55.7 |
| 3 | Compound Curing | 1 | 3-CC-01 | 29.5 |
| 3 | Compound Curing | 2 | 3-CC-02 | 33.4 |
| 3 | Colloidal Silica Sealer | 1 | 3-CSS-01 | 24.2 |
| 3 | Colloidal Silica Sealer | 2 | 3-CSS-02 | 14.3 |
| 4 | Compound Curing | 1 | 1-CC-01 | 116.1 |
| 4 | Compound Curing | 2 | 1-CC-02 | 206.4 |
| 5 | Compound Curing | 1 | 2-CC-01 | 31.5 |
| 5 | Compound Curing | 2 | 2-CC-02 | 59.0 |
| 1 | Moisture Curing | 1 | 1-MCS-01 | 0.7 |
| 1 | Moisture Curing | 2 | 1-MCS-02 | 5.2 |
| 2 | Moisture Curing | 1 | 2-MCS-01[2] | - |
| 2 | Moisture Curing | 2 | 2-MCS-02 | 2.3 |
| 3 | Moisture Curing | 1 | 3-MCS-01 | 2.5 |
| 3 | Moisture Curing | 2 | 3-MCS-02 | 2.0 |
| 4 | Moisture Curing | 1 | 4-MCS-01 | 43.0 |
| 4 | Moisture Curing | 2 | 4-MCS-02 | 68.6 |
| 5 | Moisture Curing | 1 | 5-MCS-01 | 77.2 |
| 5 | Moisture Curing | 2 | 5-MCS-02[2] | - |
| 1 | No Curing | 1 | 1-NC-01 | 16.5 |
| 1 | No Curing | 2 | 1-NC-02 | 18.6 |
| 2 | No Curing | 1 | 2-NC-02 | 29.3 |
| 2 | No Curing | 2 | 2-NC-02 | 51.6 |
| 3 | No Curing | 1 | 3-NC-01 | 3.0 |
| 3 | No Curing | 2 | 3-NC-02 | 9.7 |
| 4 | No Curing | 1 | 4-NC-01 | 197.8 |
| 4 | No Curing | 2 | 4-NC-02 | 222.0 |
| 5 | No Curing | 1 | 5-NC-01 | 92.2 |
| 5 | No Curing | 2 | 5-NC-02 | 111.7 |

Note:

[1] All the cells highlighted in orange have passed the 0.5 $kg/m^2$ limit of BNQ NQ 2621-900

[2] For 2-MCS-01 and 5-MCS-01, testing halted prematurely due to leaking

### 4.4.4 Visual Rating Results

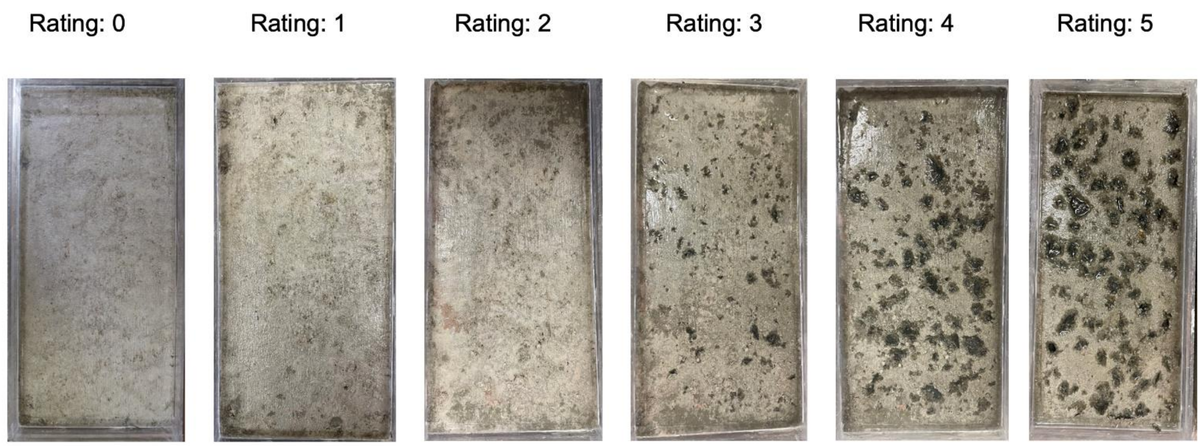



**Figure 4.8 Visual rating scale example of specimen mix 1 CC**

Figure 4.12 shows the visual rating data for all Mix 1 specimens for the two testing standards used. The specimens were visually rated according to Table 4.15 and the ASTM C672 visual rating procedure. The visual rating is taken at different cycles for ASTM C672 and BNQ NQ 2621-900. The lines in the figure represent the average visual rating of 2 specimens for both ASTM C672 and BNQ NQ 2621-900. For ASTM C672, the visual rating is taken at 5, 10, 15, 25, and 50 cycles; for BNQ NQ 2621-900, the visual rating is taken at 7, 21, 35, 56, 77, 100, and 112 cycles. The BNQ NQ 2621-900 does not require visual rating in the procedure. However, in this study, the visual rating was taken every time scaled-off mass was collected from the specimens to facilitate comparisons between the two standards.

### 4.4.5 Combined Visual Rating and Cumulative Scaled Mass Results

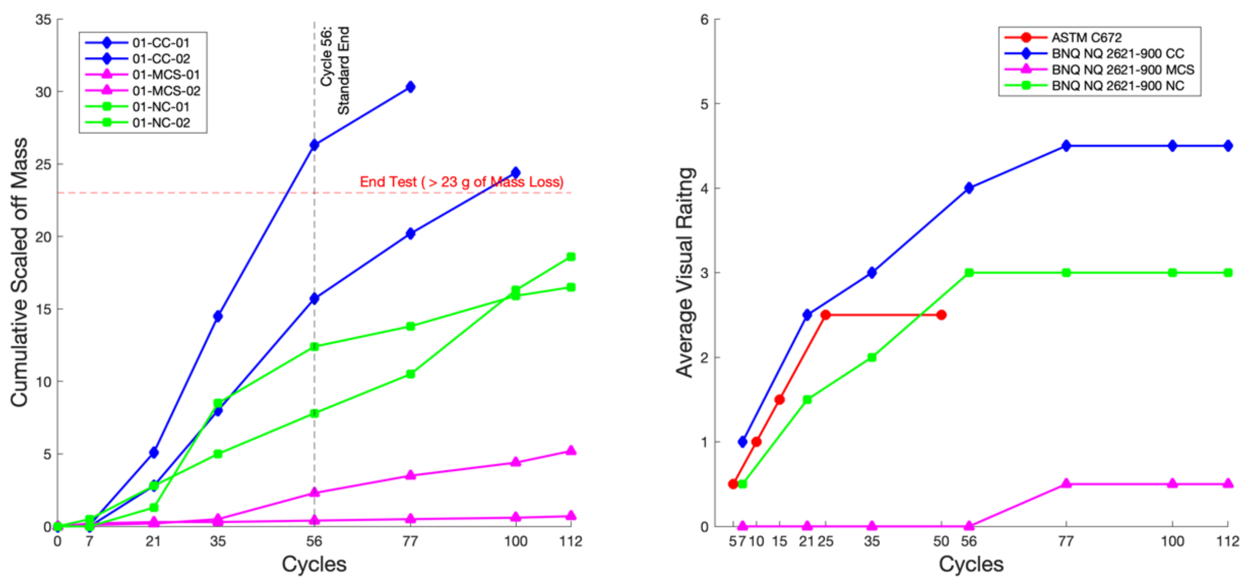



**Figure 4.9: Mix 1 comparison of scaled off mass and visual rating**

Figure 4.13 illustrates that the visual rating of ASTM C672 for Mix 1 specimens closely follows the BNQ 2621 compound cured (CC) specimens up to 21 cycles. For moisture cured

(MCS) specimens, the visual rating is 0, corresponding to collecting less than 5g of scaled-off mass after 112 cycles. This observation is valid for compound cured (CC) and no cured (NC) specimens, where the visual rating is 3 and 4, corresponding to scaled-off mass above 15 to 20g and above 25g, respectively.

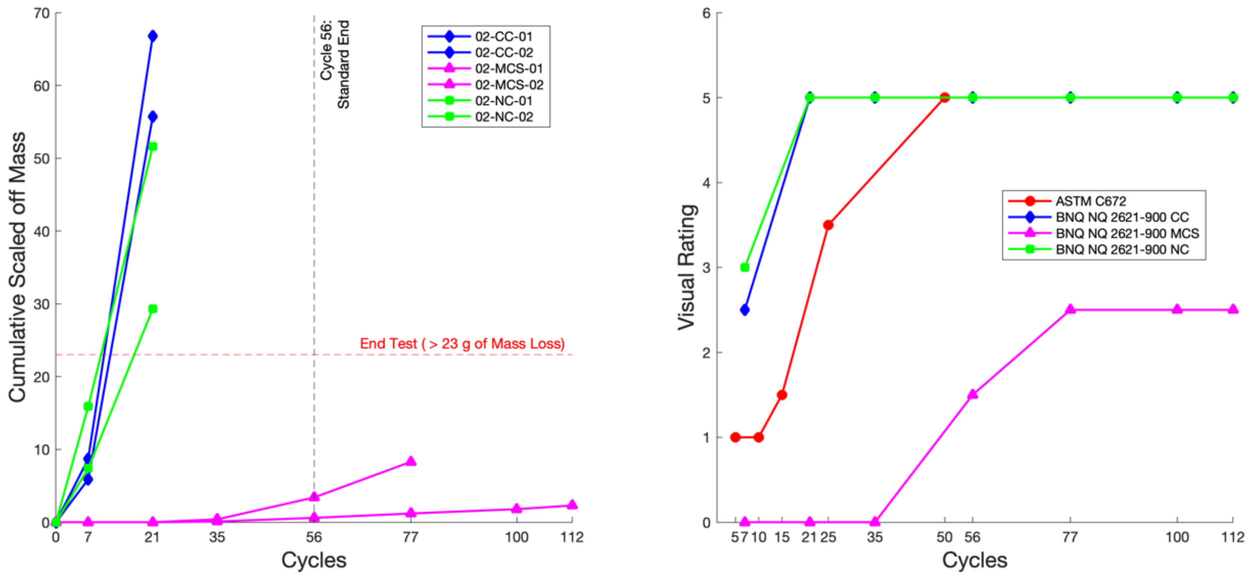



**Figure 4.10: Mix 2 comparison of scaled off mass and visual rating**

Figure 4.14 shows Mix 2 cumulative scaled off mass. All compound cured (CC) and no cured (NC) specimens failed at cycle 21 and moisture cured (MCS) passed 56 cycle test cycle. 02-MCS-01 was pulled due to brine leaking through the bottom of the specimen though 02-MCS-1 finished the test at cycle 112 under the mass loss limit. Figure 4.10 a and b, show that when compound cured (CC) and no cured (NC) specimens generate more than 23g, the visual rating is 5, corresponding to failure of the test. While moist cured (MCS) specimens generated less than 10g after 112 cycles which correspond to visual rating of 2.

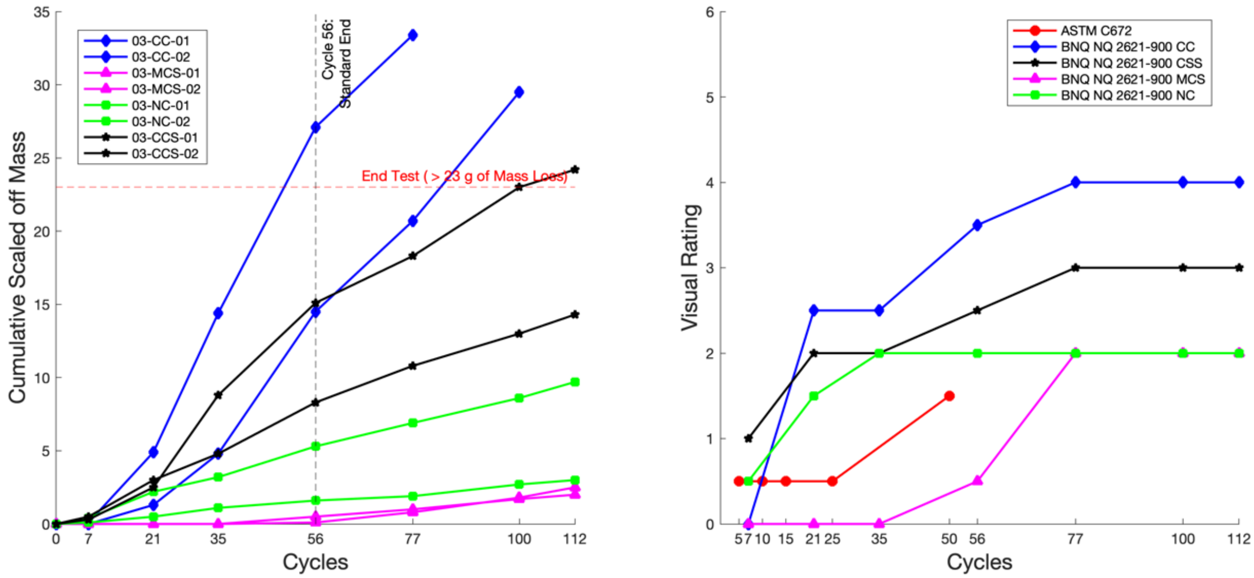



**Figure 4.11:Mix 3 Comparison of scaled off mass and visual rating**

Figure 4.15 demonstrates that all the specimens except one compound cured specimen pass the BNQ 2621 test standard at cycle 56 with less than 23 grams. All the no cured (NC) and moisture cured (MCS) passed the BNQ 2621 test at cycle 112 by generating less than 10 grams of scaled-off material, which corresponds to a visual rating of 2. After 112 cycles, both the compound cured (CC) specimens and one of the specimens subjected to colloidal silica sealer (CSS) have failed the test. It can be observed from Figure 4.15, that visual rating follows the scaled off mass plot. The collected scaled off mass after 112 cycles for 3-NC-1 and 3-MCS-1 is 3 and 2.5 respectively, which corresponds to visual rating of 2. The compound cured (CC) specimens failed after scaling 23g off of the surface with a visual rating of 4.

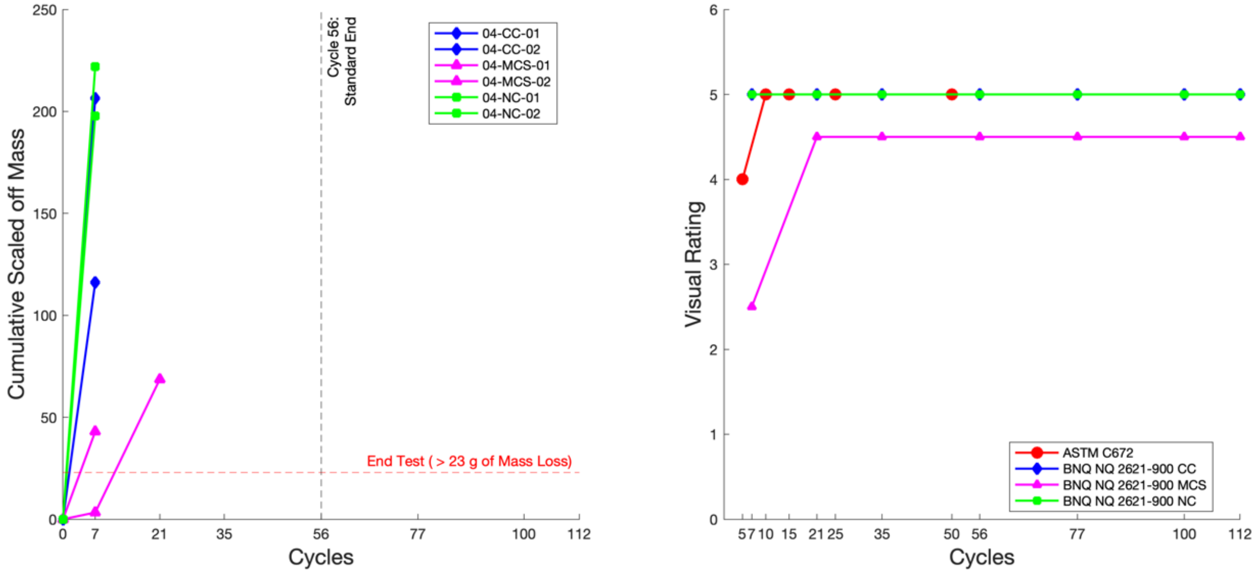



**Figure 4.12: Mix 4 Comparison of scaled off mass and visual rating**

Figure 4.16 shows Mix 4 cumulative scaled off mass. All specimens failed by cycle 21, with moisture cured (MCS) specimens generating less scaled-off material compared to compound cured (CC) and no cured (NC) specimens. For all specimens of Mix 4, the visual rating is 5, corresponding to failing the test by collecting more than 23g of scaled off mass.

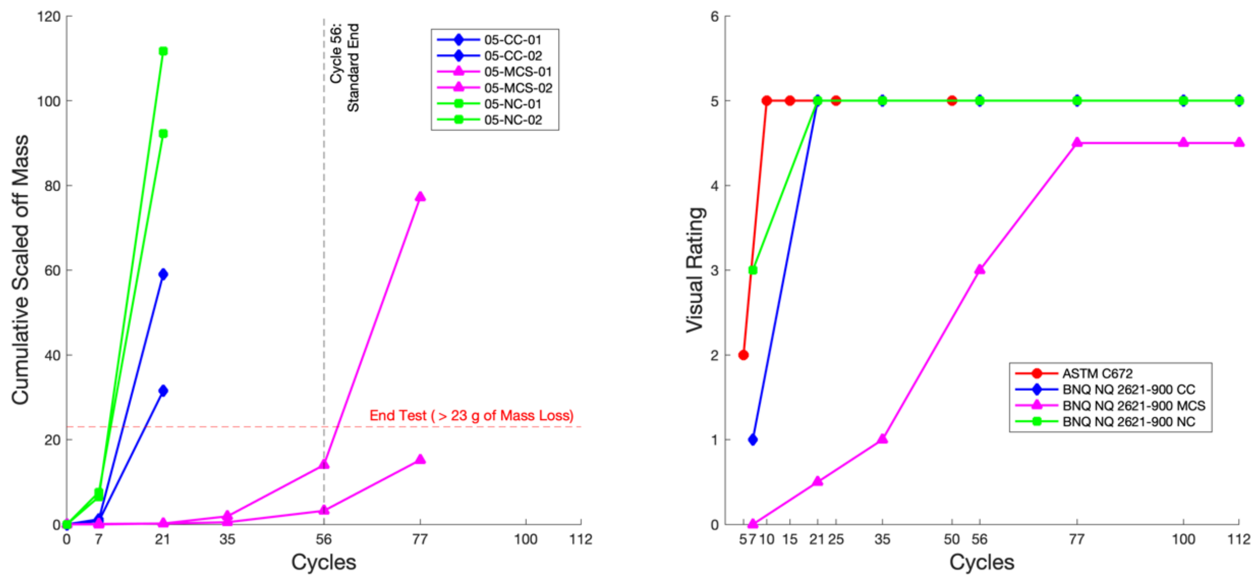



**Figure 4.13:Mix 5 Comparison of scaled off mass and visual rating**

Figure 4.17 shows Mix 5 cumulative scaled off mass. Compound cured (CC) and no cured (NC) specimens failed on cycle 21 by generating more than 23 grams of scaled-off material. Moisture cured (MCS) specimens passed the BNQ test standard with less than 23 grams of mass loss at cycle 56. However, one of the moisture cured (MCS) specimens failed by the next mass measurement and the other had severe brine seepage. The visual rating follows the cumulative scaled off mass trend which can be observed from Figure 4.13 a and b.

From Figure 4.13 and 4.17, it can be observed that the ASTM C672 visual rating closely follows the trend of compound curing (CC) and no curing (NC) in BNQ 2621. Also, the visual rating for moisture cured (MCS) specimens in BNQ 2621 test are always less than compound curing (CC) and no curing (NC) methods.

Figure 4.18 presents data of collected scaled-off mass from specimens and the assigned visual rating. Each line in the figure shows the visual rating taken when the scaled-off mass was collected until the specimens have failed the BNQ 2621 test. It can be observed that 75% of the 20 specimens which did not pass the BNQ 2621 test limit had a visual rating of 4 or 5. The visual rating 4 and 5 corresponds to scaled-off mass ranging from 24.2g to 222g. The rest of the failed specimens (out of 20) had a visual rating of 3.

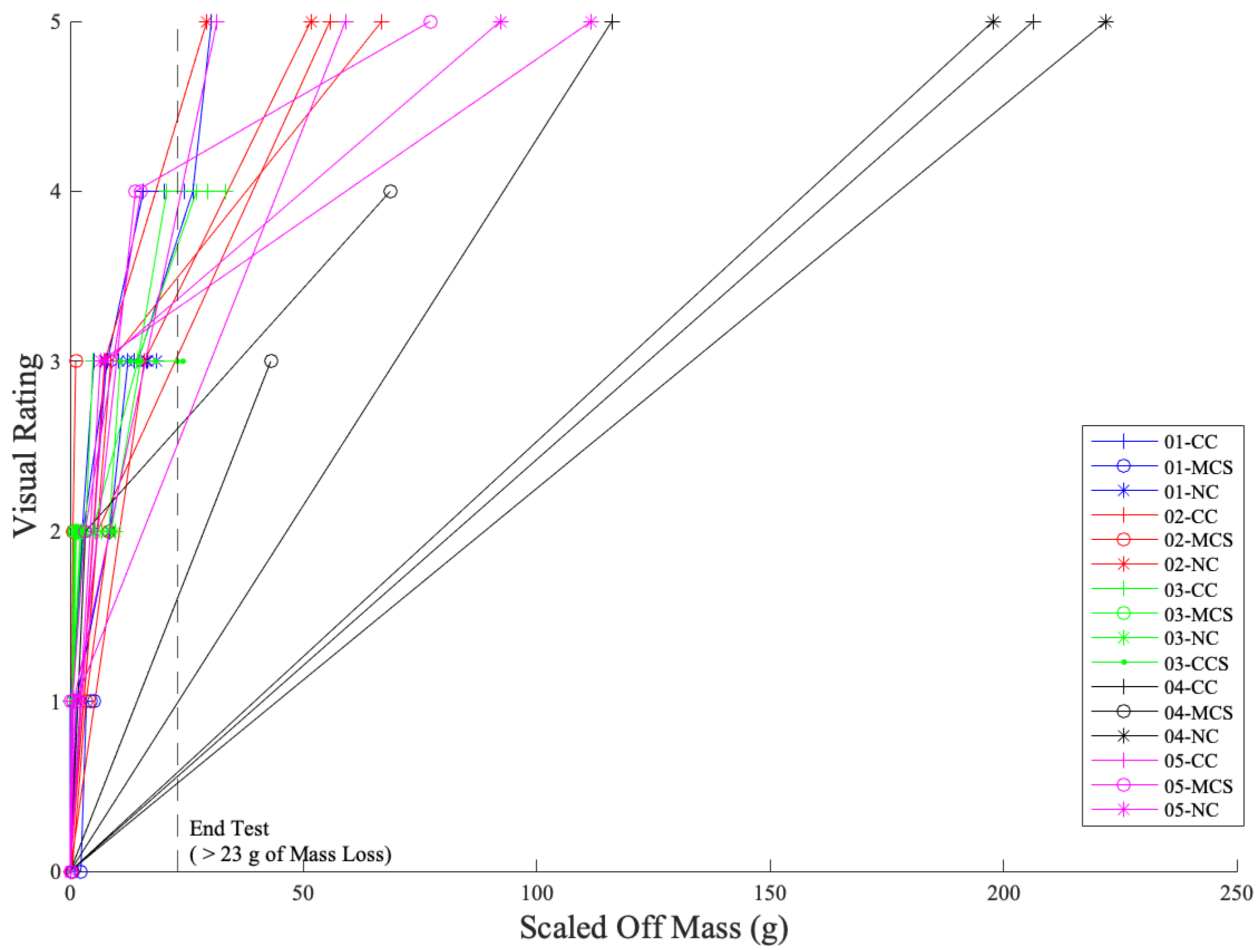



**Figure 4.14: Scaled off mass and visual rating of all specimens**

Figure 4.19 shows all specimens from BNQ NQ 2621-900 scaling resistance test by mix design. The specimens of Mix 4 all failed the test at or before 21 cycles. While most of the Mix 1 and 3 specimens passed the testing limit if 23g at cycle 112. It can be observed from the Figure 4.19 that mix design has an impact on the scaling resistance of concrete. In Mix 5, failure of specimens varied based on curing method.

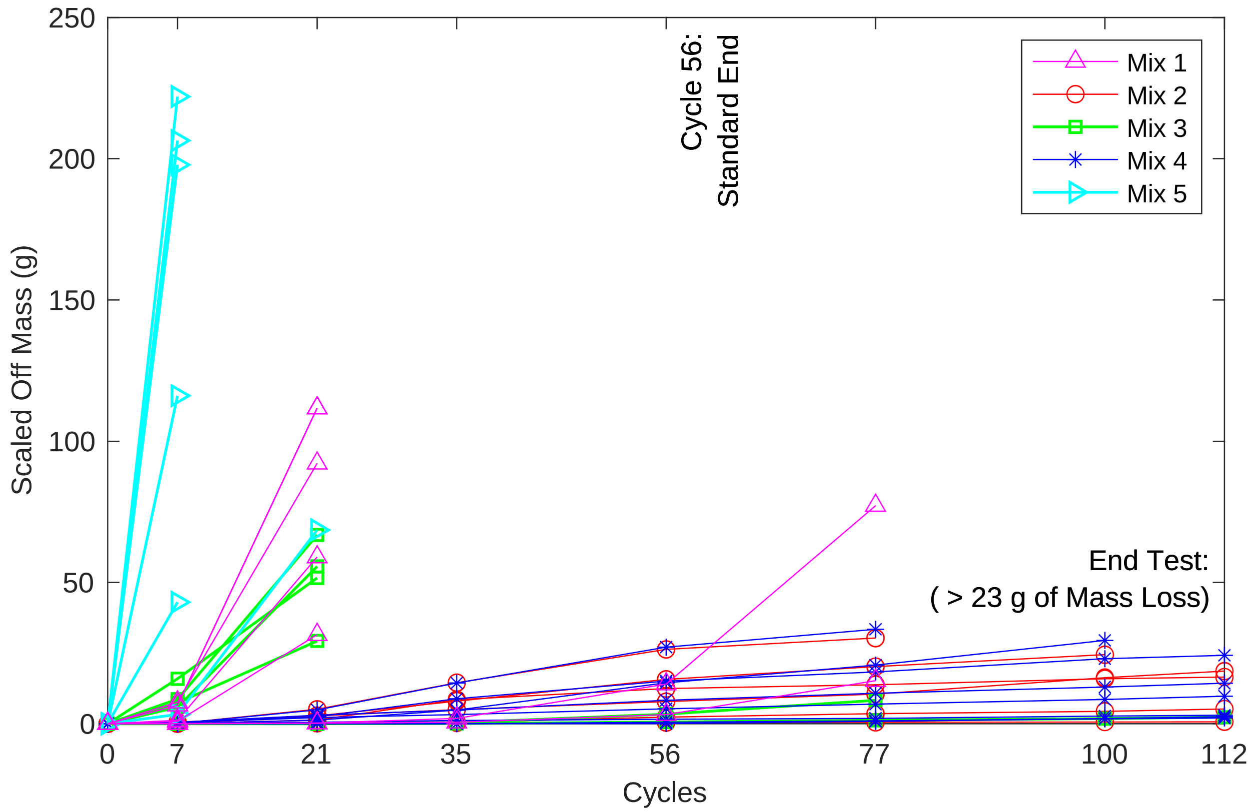



**Figure 4.15: Scaled off mass by mix**

From Figure 4.20 it can be observed that curing methods alone do not impact the scaling performance in laboratory testing. The specimens which were moisture cured (MCS) generally scale less than no cured (NC) and compound cured (CC), though a few moisture cured (MCS) have failed the BNQ 2621 standard as well. Both no cured (NC) and compound cured (CC) specimens are highly variable, indicating that curing type is an imperfect predictor of scaling performance

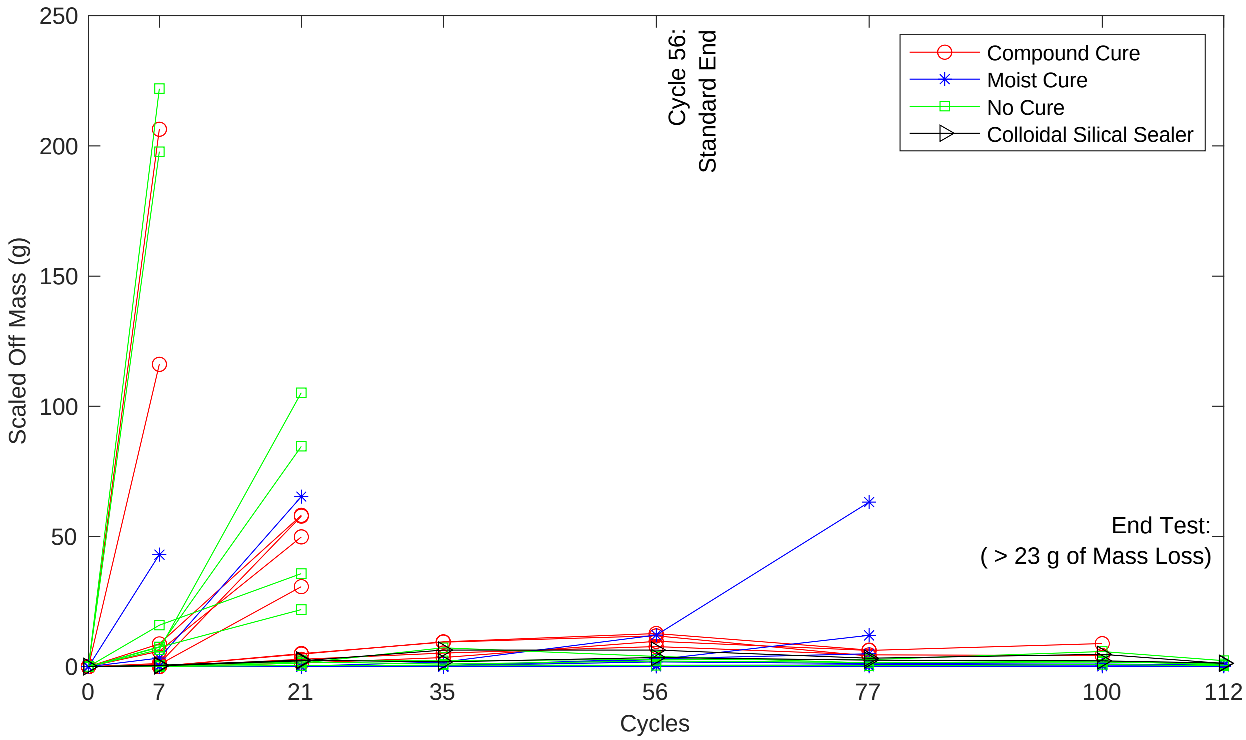



**Figure 4.16:Scaled off mass by curing method**

From Figure 4.21, Mix 4 specimens demonstrated severe scaling irrespective of curing method. Among the Mix 2 specimens, only the specimens which were moisture cured (MCS) have passed the test. Similarly, among Mix 5 specimens, the moisture cured specimens (MCS) have shown better scaling performance after 56 cycles. For both Mix 1 and 3, all specimens passed the test at 56 cycles except for 01-CC and 03-CC specimens. Among Mix 1 and 3, moisture cured (MCS) and no cured (NC) specimens have demonstrated better scaling performance compared to compound cured (CC) specimens at cycle 112.

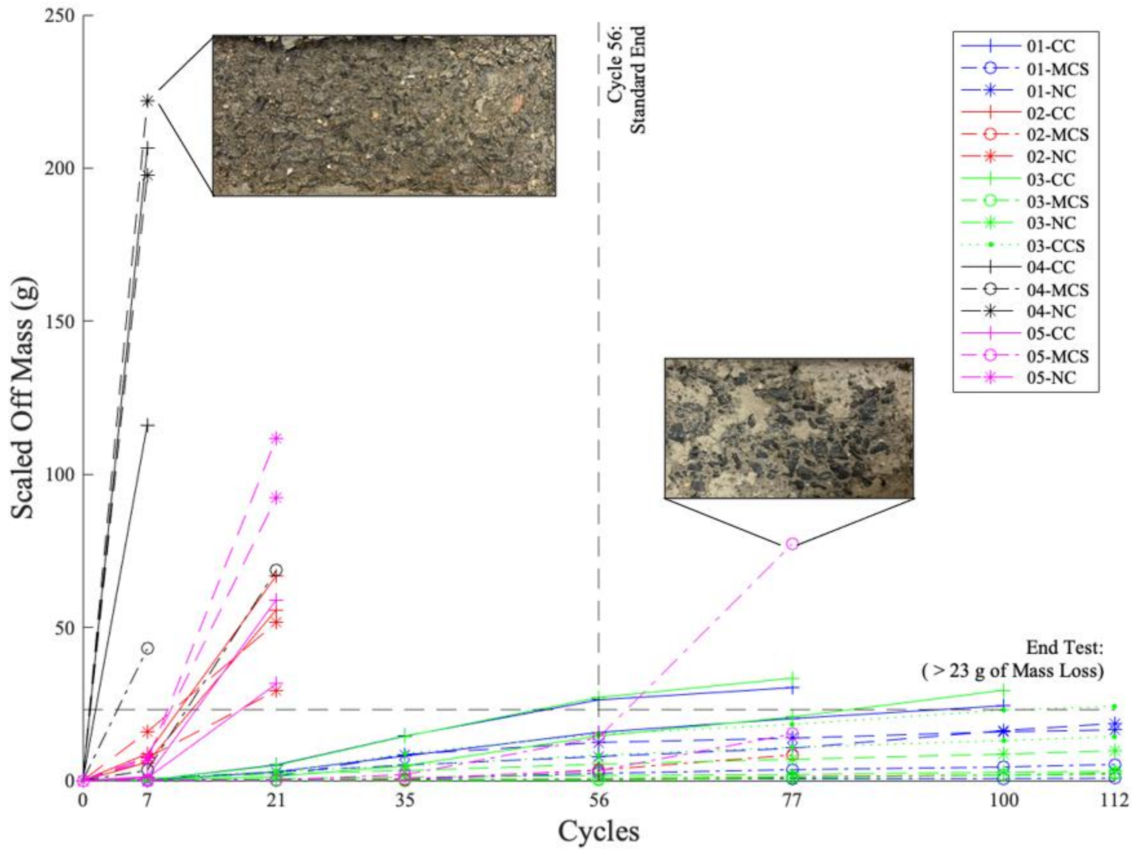



**Figure 4.17: Scaled off Mass Combined Results of Scaled-off Mass**

## 4.5 Temperature Study – Ambient Temperature and Temperature in the Concrete Sidewalk

One of the key factors that influences surface scaling of concrete is the number of freeze-thaw cycles that the concrete experiences, particularly near its surface. In the past, ambient temperature was used to estimate the number of freeze-thaw cycles in a winter season. But, the concrete temperature through thickness within a sidewalk may differ from ambient temperature. To assess the difference in concrete temperature and ambient temperature, both were recorded for 120 days during the 2021-22 winter season. An environmental meter was installed next to the sidewalk panels at Brack Laboratory to record the ambient temperature. The environmental meter (Kestrel 5200) is a handheld weather station that records temperature, wind speed, absolute pressure, air density, humidity, wind chill, and other parameters. Along with the environmental meter, thermal sensors were embedded in 12 in. by 12 in. by 6in. thickness test concrete panels to determine the temperature inside a body of concrete of approximately equal thickness to the sidewalks in this study. Thermal sensors were embedded in these concrete panels constructed using the same mix designs and constituents as the sidewalks to evaluate the difference between concrete internal temperature and ambient temperature. An additional benefit of the study is to determine whether mix design had any noticeable effect on internal concrete temperature.

The thermal sensors used to measure internal concrete temperature were Giatec SmartRock. Because these sensors were developed to determine concrete maturity, they only have a memory of 60 days. Therefore, two sets of panels were prepared for each mix to record temperatures from December 2021 to March 2022, a period that exceeded the 60-day life of each sensor. The first set of panels were prepared on December 3, 2021 and a second set on January 24, 2022. The sensors record two values: sensor body temperature and cable temperature. The sensor body was attached to a bar chair at 2" from the surface, and the cable was extended to the center of the panel at 3" from the top surface. Figure 4.22 shows the sensor setup before the formwork was filled with concrete. The same mix design and materials were used in these panels and the sidewalks. After placement, the panels were demolded after 24 hours and were cured for seven days in a curing chamber. The panels were placed next to concrete sidewalks at Brack Lab.

**Figure 4.18 Thermal sensor in mold before concrete pour**

Figure 4.23 shows temperature readings from the environmental meter and thermal sensors for each of the concrete mixes. One freeze-thaw cycle consists of freezing below 0 C for 16+ hours and above zero 8+ hours. The concrete sidewalks were subjected to 18 freeze-thaw cycles in winter.

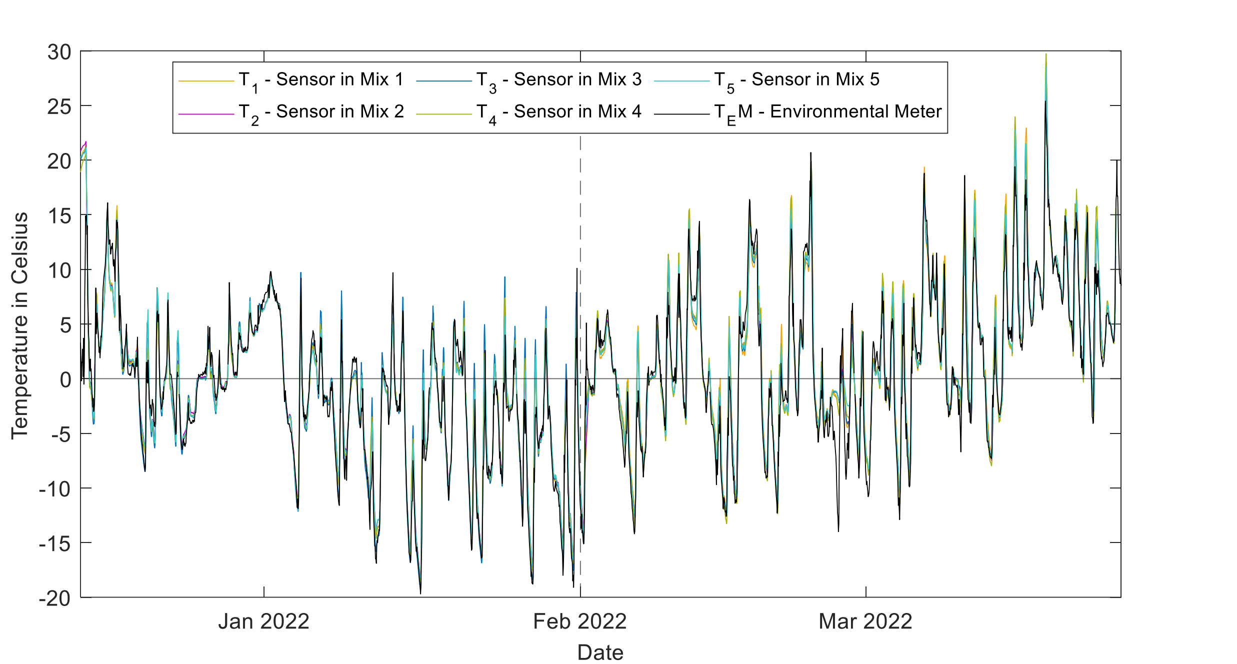



**Figure 4.19 Comparison of temperature data from December 2021 to March 2022 measured using the environmental meter and thermal sensors**

The data from meter and sensors for different mixes follow a similar path to ambient temperature except on a few occasions. From Figure 4.24, it can be seen that the temperature from the environmental meter from February 25 to February 27 is lower than the temperature inside the concrete panels by 71.43%. During these days, 6 inches of snow accumulated on concrete panels due to a snowstorm. The snow might have provided insulation to the concrete panels, resulting in higher temperatures inside concrete than the ambient temperature. The results from the temperature study indicated that ambient temperature can be used as a measure of surface concrete temperature.

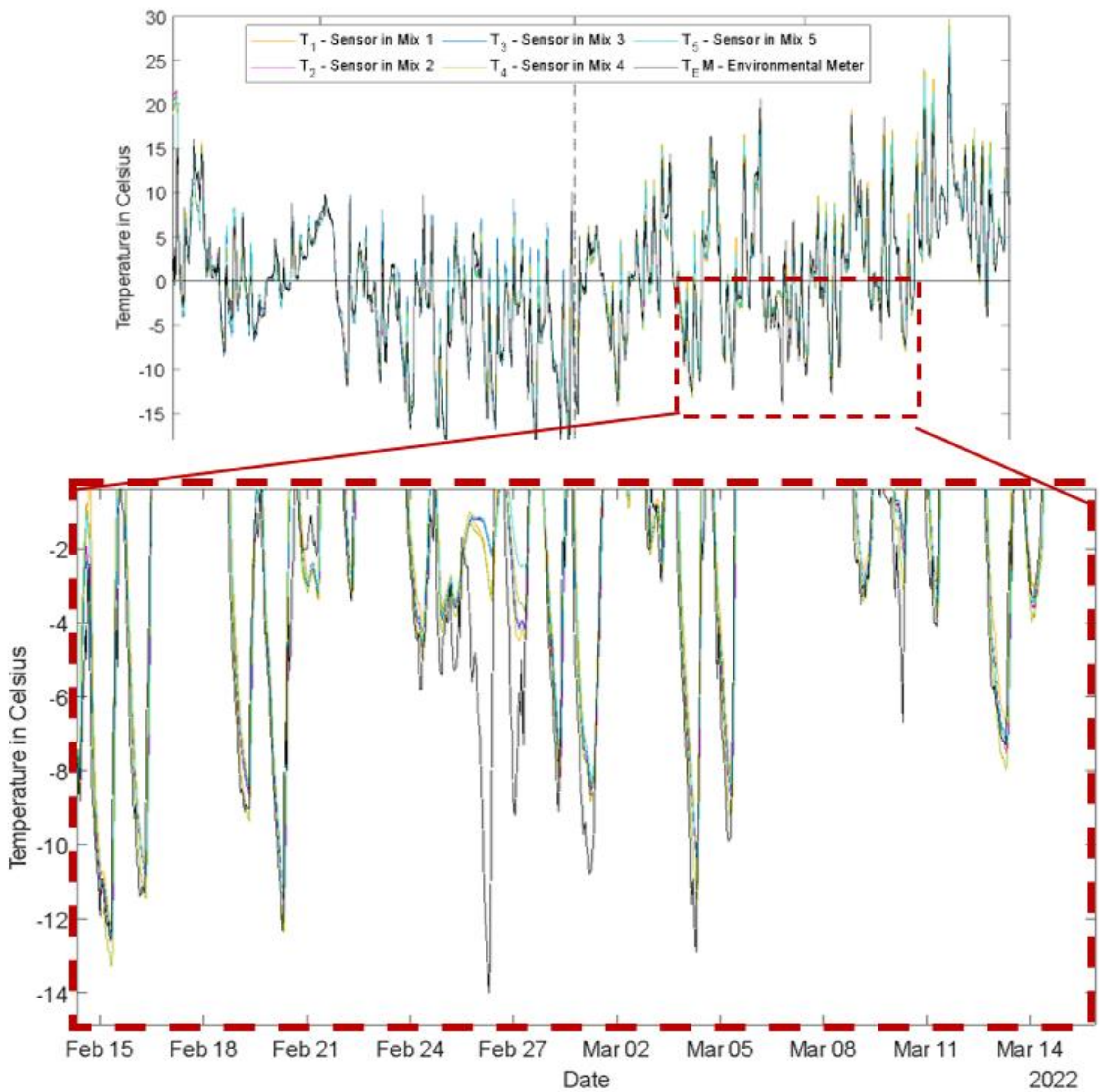



**Figure 4.20 Temperature data from February 25th to 27th when ambient temperature is lower than temperature inside concrete due to blanket of snow on concrete panels**

In January 2021, the temperature recorded by the sensor in Mix 3 had higher peaks than other sensors on several occasions, as shown in Figure 4.25. The temperature inside Mix 3 recorded was, on an average of, 89.35% higher than Mix 4. Similarly, on an average, Mix 3 peaked over Mix 5, Mix 2, and Mix 1 by 151.65%, 158.44%, and 103.77%, respectively.

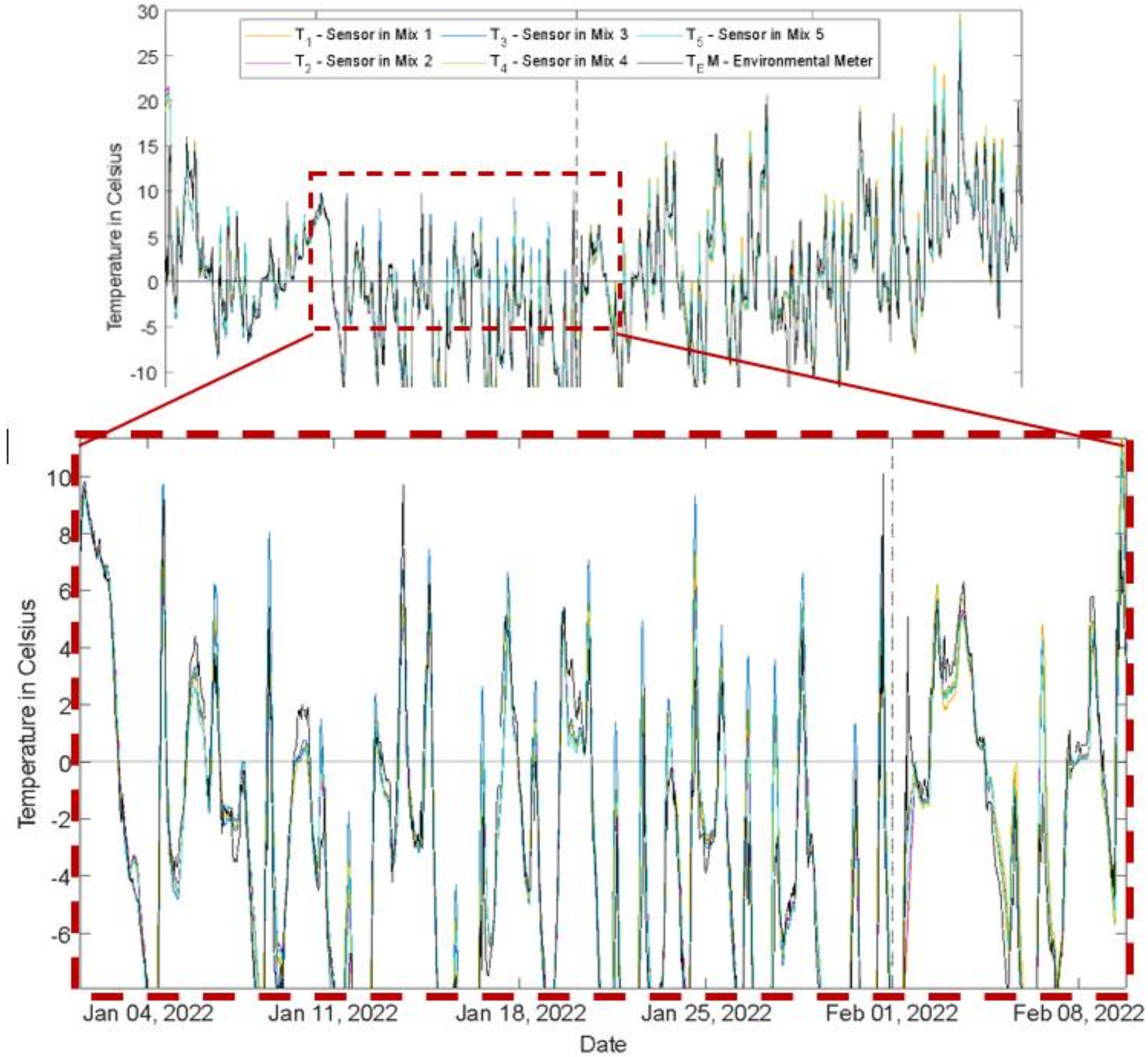



**Figure 4.21 Temperature data in January 2021 where the temperature of Mix 3 specimens is greater than other mix designs**

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# 5.0 Construction Practices

The UMass Research and MassDOT Research and Materials groups oversaw the pre-placement, placement, and finishing practices involved in the project. The contractor followed the best pre-placement, placement, finishing, and curing practices as described in Chapter 2.

Panel Groups A, through C were placed on the western side of the south yard at Brack Laboratory, while Panel Groups D through F and G through I were placed in two rows, respectively, on the east side of the storage yard. All panels were placed to approximately follow the yard grade in the longitudinal direction of each row. Transversely, panels in Groups A through C had a 1.5% cross-slope towards the laboratory yard, and panels in groups D through F and G through I were built with a 1.5% cross-slope to provide drainage away from the adjacent panel row.

Individual concrete trucks were used to supply each different mix; therefore, sidewalk panels were placed by mix. It was intended to deliver concrete mixtures 1, 3, and 5 on 26 July 2021 and mixtures 2 and 4 were delivered on 27 July 2021. However, a second batch of Mix 3 was also delivered on 27 July because of an error in volume needed to place all the panels needed with Mix 3.

## 5.1 Pre-Placement, Placement, and Finishing Practices

### 5.1.1 Pre-Placement

The sitework for the project began the week of 19 July 2021 at the Brack Structural Engineering Laboratory (Brack Lab) at the University of Massachusetts Amherst.

Baltazar Contractors laid out the site and performed the excavation for three sidewalk rows. Following procedures described in Chapter 2, Figure 5.1 shows the excavation and compacted subgrade before the formwork, or the subbase were added.

**Figure 5.1 Panels groups A, B, and C after excavation (looking south)**

Caracas Construction constructed the formwork and placed concrete for the sidewalk panels. Figure 5.2 shows the formwork in place with the subgrade compacted before the subbase was placed.

**Figure 5.2 Panel groups D to I (looking south) with framing before placement**

### 5.1.2 Placement

Concrete placement took place on 26 and 27 July 2021, when high ambient temperatures and high humidity were forecasted, as required for the study.

**Figure 5.3 Concrete coming down the chute of concrete truck into panel frame**

Temperature and weather conditions during the placement days can be found in Table 5.1.

**Table 5.1 Weather data from 26 July 2021 and 27 July 2021**

| Mixture | Date | Action | Time | Temperature (°F) | Relative Humidity (%) | Heat Index (°F) | Dew Point (°F) | Air Speed (mph) | Evaporation Rate (lb/ft²/h) |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 7/26 | Arrival | 10:04 | 85 | 67.5 | 93 | 73 | 0 | 0.01 |
| 1 | 7/26 | Depart | 10:51 | 85.3 | 58.8 | 90.9 | 69.2 | 0 | 0.02 |
| 2 | 7/27 | Arrival | 10:02 | 88.8 | 44.6 | 91.9 | 64.5 | 0 | 0.02 |
| 2 | 7/27 | Depart | 10:38 | 97.4 | 48.5 | 112.8 | 74.7 | 0 | 0.01 |
| 3 | 7/26 | Arrival | 11:22 | 83.8 | 55.5 | 86.7 | 66.2 | 2.1 | 0.04 |
| 3 | 7/26 | Depart | 12:10 | 87.6 | 50.5 | 91.8 | 66.9 | 1.4 | 0.03 |
| 3 | 7/27 | Arrival | 1:02 | 97.9 | 38.4 | 106.7 | 68.2 | 0 | 0.02 |
| 3 | 7/27 | Depart | 1:37 | 97.4 | 33.9 | 102.7 | 64.3 | 0 | 0.02 |
| 4 | 7/27 | Arrival | 11:26 | 86.7 | 47.1 | 89.2 | 64.1 | 0 | 0.02 |
| 4 | 7/27 | Depart | 12:00 | 90.8 | 45.2 | 96.1 | 66.7 | 0 | 0.02 |
| 5 | 7/26 | Arrival | 1:05 | 87.3 | 56.1 | 93.7 | 69.7 | 1.7 | 0.03 |
| 5 | 7/26 | Depart | 1:50 | 91.1 | 44.1 | 96.3 | 66.2 | 1.3 | 0.03 |

Nonconforming procedures:

- NRMCA's Hot Weather Concreting allows between 2-2.5 gallons of water per cubic yard of concrete. Since the mixtures were 5 cubic yards, a maximum of 10-12.5 gallons of water can be added per mixture:

    Based on the placement data sheets additional water added in transit or at the site ranged from 7-15 gallons.

    - Mixture 1 added 3 gallons in transit and no additional water was added at the site.
    - Mixture 2 added 15 gallons on site, which is out of conformity.
    - Mixture 3 placement 1 added 10 gallons at the concreting facility and 7 at the site and placement 2 added 10 gallons at the site.
    - Mixture 4 has 15 gallons added at the site
    - Mixture 5 had 15 gallons of water added at the site.

    Mixtures 2, 3 placement 1, 4, and 5 were non-conforming.

### 5.1.3 Finishing

Finishing began after bleed water was not present on the surface of the sidewalk panels. The Thumb-Press method was used to determine whether or not bleed water was present. The thumb is pressed against the surface of the concrete and if water does not appear in the impression, then finishing may begin. Finishing practices conformed to procedure as described in Chapter 2 with exception to the foot and lumber imprints left on the top surface of several panels. Bull Float, Trowel, and Brush shown in Figures 5.4a, 5.4b, and 5.5 show the appropriate use of the tools.

Nonconforming procedures:

- Footprint and lumber imprints were left on the surface of the several panels.

As listed in Table 5.2, finishing was completed between 0.5 to 1.5 hours after placement concluded. The panels were typically bull floated, sprayed with an evaporation reducer if stipulated, bull floated again, troweled, and then broomed with pauses as necessary.

**Table 5.2 Finishing conditions**

| Placement | Date | Avg. Temperature (°F) | Avg. Relative Humidity (%) | Weather Conditions | Mix No. | *Finishing* Completion After Placing (Hrs) |
|---|---|---|---|---|---|---|
| 1 | 7/26 | 86.7 | 55.4 | High Ambient Temperatures, High Humidity, Clear Skys, and Sunny | 3 | 0.5 - 1.0 |
| 1 | 7/26 | 86.7 | 55.4 | High Ambient Temperatures, High Humidity, Clear Skys, and Sunny | 1 | 1.0 |
| 1 | 7/26 | 86.7 | 55.4 | High Ambient Temperatures, High Humidity, Clear Skys, and Sunny | 5 | 0.5 - 1.0 |
| 2 | 7/27 | 93.2 | 43.0 | High Ambient Temperatures, High Humidity, Clear Skys, and Sunny | 2 | 0.5 - 1.0 |
| 2 | 7/27 | 93.2 | 43.0 | High Ambient Temperatures, High Humidity, Clear Skys, and Sunny | 3 | 0.5 |
| 2 | 7/27 | 93.2 | 43.0 | High Ambient Temperatures, High Humidity, Clear Skys, and Sunny | 4 | 1.0 - 1.5 |

**Figure 5.4 Workers use bull float and trowel on sidewalks**

**Figure 5.5 Worker uses trowel on concrete panel**

**Figure 5.6 Worker using brush push broom to finish the surface of the panel**

## 5.2 Curing Methods

The saturated cover, no cure, and the liquid membrane forming compound curing methods were applied following the procedures in Section 2.4

Panel Groups A, D, and G were cured and sealed using an evaporation reducer and an ASTM C1315 Curing/Sealing Compound. Mixture numbers 1, 2, 3, 4, and 5 in panel groups B, E, and H were cured and sealed using an evaporation reducer, subjected to saturated cover curing, and penetrating sealer. Panel mixture number 3-CSS in panel groups B, E, and H were cured and sealed using an evaporation reducer and colloidal silica sealer method. panel groups C, F, and I were cured with air curing without an evaporation reducer or sealing.

Temperature and relative humidity were measured next to the curing panels for 7 days following placement and are displayed in Figures 5.6 and 5.7, respectively. According to the NRMCA, low relative humidity, high winds, or solar radiation can cause the same effects as high temperature on a curing concrete specimen by causing evaporation of liquid. Furthermore, high relative humidity can help diminish the effects of high temperatures on concrete.

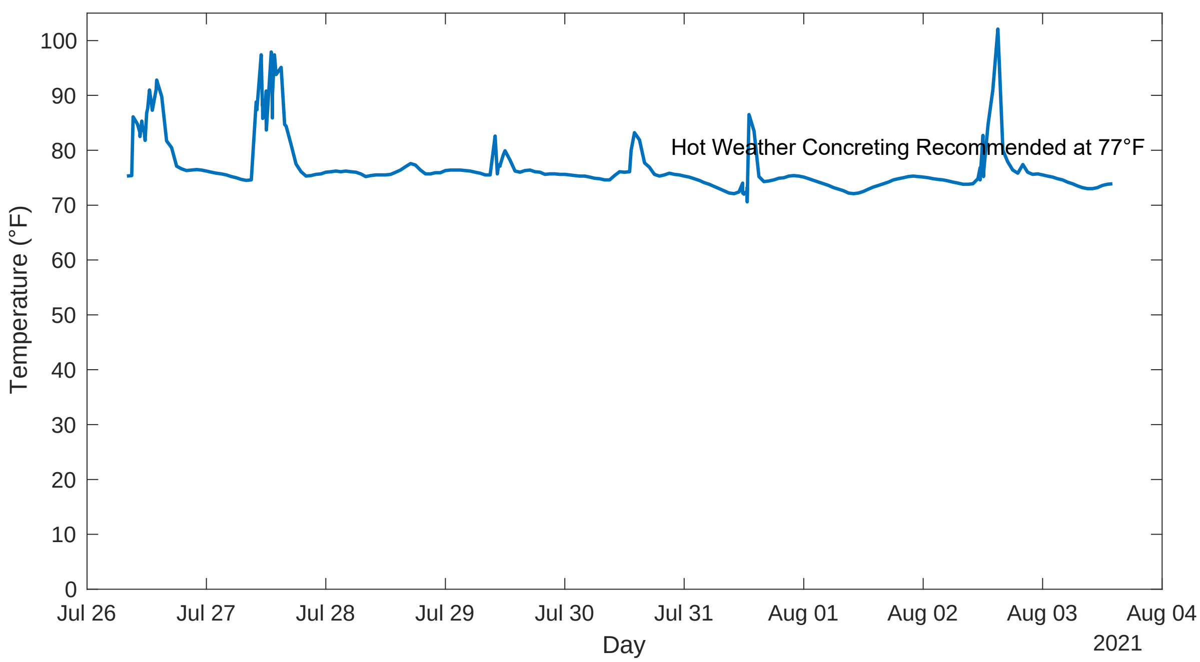


**Figure 5.7 Temperature over 7 days after placement**

Figure 5.6 shows a temperature minimum for which hot weather concreting practices is recommended (77 °F or above).

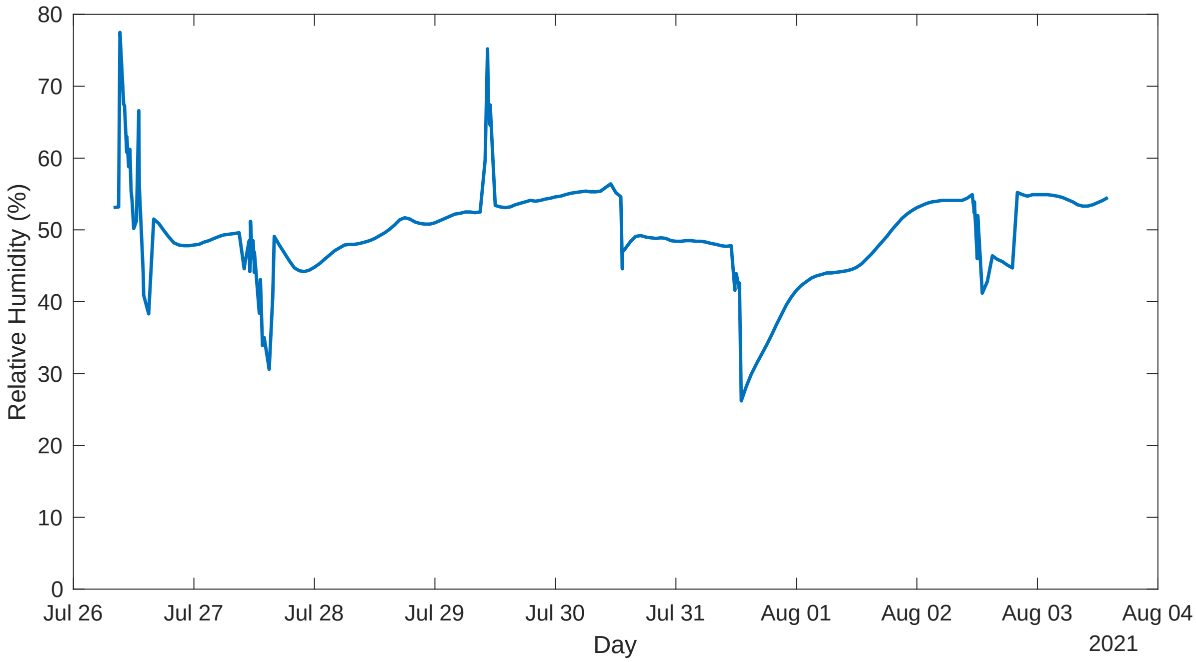


**Figure 5.8 Relative humidity over 7 days after placement**

Figure 5.8 shows the relative humidity for the 7 days following placement of the sidewalk specimens. A high relative humidity (80 – 90 %) during placement has been observed to induce scaling in concrete sidewalks. The relative humidity is not measured in this range during the 7 days following placement, but is observed to be nearing the lower limit on two occasions.

### 5.2.1 Saturated Cover Application

In Figure 5.6 the plastic covering is placed over the burlap cover to keep the concrete surface moist for the moist curing. The application of the saturated cover conformed to the procedure described in section 2.4.1 except stated below. The burlap was covered with a plastic sheet and saturated daily for 5 days to remain moist.
Nonconforming procedures:

- Burlap caused discoloration and stains on the panels.

**Figure 5.9 Panel groups D-E-F and G-H-I after formwork was removed with saturated covering**

### 5.2.2 Liquid Membrane Forming Compound for Curing and Sealing Application

An ASTM C1315 Type 1 Liquid Membrane Forming Compound (Cure Shield EX by SpecChem) was applied to panel groups A, D, and G following the materials and application provisions of Section 2.4.2. This liquid membrane forming compound cures and seals by forming a membrane over the surface of the concrete and is best applied to a damp surface. The fugitive red dye can be seen in Figure 5.6 and the manufacturer stated it would disappear after time.

### 5.2.3 No Curing

No Curing concreting panels were exposed to ambient weather conditions displayed in Figures 5.6 and 5.7. The panels received no evaporation reducers, curing compounds, or saturated cover.

**Figure 5.10 Panel group 5H is covered in a moist cover and 1I is uncovered with no curing methods**

# 6.0 Petrographic Study

For the extracted cores from the concrete sidewalk panels, the petrographic examination was performed by Wiss, Janey, Elstner Associates, Inc. (WJE) at their laboratory in Northbrook, Illinois. The comments in this section are either direct excerpts or interpretation of comments from WJE report *Laboratory Studies of Twenty-One Concrete Cores* (Appendix – full report as received from WJE). On the 21 concrete cores extracted from several sidewalk panels at UMass Robert Brack Structural Engineering Laboratory, WJE performed petrographic studies, air void system analysis, and determined chloride ion content. WJE conducted petrographic studies and outlines their comments in an interim report dated October 25, 2022.

On June 13, 2022, Janney Technical Center of Wiss, Janey, Elstner Associates, Inc. (WJE) in Northbrook, Illinois received twenty-one cores full depth concrete cores of 4-inch nominal diameter. These cores represent five different concrete mixture designs which are summarized in Chapter 3. The concrete mix designs along with specific gravities of the materials and timing of application of curing compound were provided to WJE for review, information, and comparison purposes. Plastic air contents during placement were provided by UMass. However, air contents were not included in the mixture designs. Therefore, WJE estimated air contents by assuming that provided quantities of materials yielded 1 cubic yard of concrete. The petrographic summary for each core along with sample ID and corresponding mix design designations are summarized in Table B-1 through Table B-5 (Appendix B of WJE Report). The images of each core are shown in datasheets (Appendix E of WJE report).

As reported in WJE report:

> The surface condition of the cores generally ranged from an intact broom-finished surface with no observable surface loss, to minor to severe mortar flaking, to moderate to severe surface loss from scaling. The depth of the scaling appeared to be superficial and was generally within the upper few millimeters of paste. The bottom surfaces of the cores were uneven and generally exhibited adhered base 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.
> Petrographic studies were conducted in accordance with ASTM C856, *Standard Practice for Petrographic Examination of Hardened Concrete (12)*. Photographs of the cores, as received in WJE's laboratory, were taken before sample preparation (see photos in the datasheets for each sample in Appendix E). Nominal ¾ inch to 1 inch thick slabs were cut longitudinally from approximately the mid-section of each core using a water-cooled, continuous-rim, diamond saw blade. One resulting planar saw-cut surface of each slab was lapped using progressively finer silicon carbide free abrasives to achieve a fine, matte finish suitable for examination with a stereomicroscope. Lapping exposes textural features such that characteristics of the paste, aggregate, and

air void system can be more easily observed microscopically. Lapped surfaces of the cores can be seen listed with the mix designs in Appendix B of WJE report and in the attached datasheets for each core in Appendix E of WJE report. Fresh fracture surfaces were also prepared to study the physical properties of the concrete and for the purposes of measuring carbonation depth from the top surfaces of the cores using phenolphthalein pH indicator solution. A copper probe was used to qualitatively assess paste hardness on laboratory-induced fresh fracture surfaces. A blue-dye epoxy-injected thin section was prepared encompassing the top surface of each core to further assess the microstructure of the paste. The thin sections were examined at magnifications ranging from 3.6X to 630X using a petrographic (polarized light) microscope.

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 showing the measured parameters of the air void systems are provided in Table C-6 through Table C-10 in Appendix C of 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 5 and 9 in. long. 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 in. from the top surface, and the other slice sampled between 4-1/2 and 4-3/4 in. 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 total water-soluble chloride ion content was measured as a percent by weight of sample (percent by weight of concrete) for each powder sample. Concentrations of water-soluble chloride ion as a percent by weight of cement and by weight of cementitious material were estimated based on the reported mixture designs. The measured and calculated chloride ion concentrations are presented in Appendix D of WJE report.

## 6.1 Examination of Core Surface

The chloride ion content was determined for extracted core surface. The amount of chloride ions on the surface are from external sources like deicing chemicals. This amount is directly related to the scaling resistance of the concrete surface.

Table 6.1 shows the water-soluble chloride ion content of concrete for all twenty-one cores. It can be observed that the cores which were subjected to NaCl had the highest amount of chloride ion content on the surface. The core with the highest amount of chloride ion content on surface, had severe scaling on the surface. This core corresponds to Mix 4 which was considered poor mix (no air entrainment, excess fine aggregates) and was subjected to no curing. It should be noted that for the two cores extracted from sidewalk with Mix 1 and no curing, the core with chloride ion content of 25 ppm had showed signs of mortar flaking on its surface. However, the other core with chloride ion content of 110 ppm had no signs of scaling or mortar flaking. Similarly, the core extracted from sidewalk with Mix 4 subjected to NaCl and cured with a chemical compound, showed no signs of scaling or mortar flaking but had a chloride ion content of 380 ppm. Other Mix 4 cores with different curing methods and subjected to NaCl, with chloride ion contents of 380 and 390 ppm had shown signs of sever mortar flaking and scaling.

**Table 6.1 Chloride ion content results from petrographic analysis of cores**

| Panel Group | Deicing Method | Curing Method | $Cl^-$ ppm Mix 1 | $Cl^-$ ppm Mix 2 | $Cl^-$ ppm Mix 3A | $Cl^-$ ppm Mix 3B | $Cl^-$ ppm Mix 4 | $Cl^-$ ppm Mix 5 | Avg. Curing Method | Avg. Deicing Method |
|---|---|---|---|---|---|---|---|---|---|---|
| A | NaCl | ASTM C1315 Curing and Sealing Compound | 60 | - | - | 140 | 380 | - | 193 | 145 |
| B | NaCl | Saturated Cover | - | 30 | - | - | 380 | - | 205 | 145 |
| B | NaCl | Colloidal Silica Sealer | - | - | - | - | - | - | - | 145 |
| C | NaCl | No Curing | 25 | 25 | 25 | - | 390 | - | 100 | 145 |
| C | NaCl | No Curing | 110 | 25 | 25 | | 390 | | 100 | 145 |
| D | $MgCl_2$ | ASTM C1315 Curing and Sealing Compound | - | 25 | - | - | - | 25 | 25 | 25 |
| E | $MgCl_2$ | Saturated Cover | - | - | 25 | - | - | - | 25 | 25 |
| E | $MgCl_2$ | Colloidal Silica Sealer | - | - | 25 | - | - | - | 25 | 25 |
| F | $MgCl_2$ | No Curing | - | - | 25 | - | - | - | 25 | 25 |
| G | Blended Brine* | ASTM C1315 Curing and Sealing Compound | 25 | - | 25 | - | - | - | 25 | 25 |
| H | Blended Brine* | Saturated Cover | 25 | - | - | - | - | 25 | 25 | 25 |
| H | Blended Brine* | Colloidal Silica Sealer | - | - | - | - | - | - | - | 25 |
| I | Blended Brine* | No Curing | - | - | - | - | - | 25 | 25 | 25 |

*Blended Brine: 85% NaCl + 15% $MgCl_2$

WJE noted that most of the cores had chloride concentrations at or near the lower end of the range in the top slice between ¾ and 1 inch of top surface. For the bottom slices between 4-

1/2 to 4-3/4 in. from the top surface, the chloride concentrations were below the detection limit of the test. It should be noted that currently, the chloride ion concentrations are very low when compared with threshold values used to determine risk of steel reinforcement corrosion although the panels did not contain any reinforcement), excepting the top surface of Mix 4 cores. The chloride concentrations are expected to rise over time due to repeated application of salts during winter months. The chloride concentrations are also expected to rise due to breakdown of sealers unless sealers are reapplied before losing effectiveness. The accumulation and buildup of chloride ion concentrations in concrete is directly related to the permeability of concrete.

### 6.1.1 Surface Distress

As summarized in WJE report, the twenty-one cores examined in the studies had no, weak, absorptive layer near the top surface. In contrast, the previously examined 60 cores for Phase- I, which showed a weak top surface layer. The twenty-one cores occasionally exhibited early-age drying shrinkage cracks due to loss of surface moisture. The surface moisture may be lost due to either evaporation of water from surface faster than it is replenished by bleed water, and/or inadequate curing that allows surface to dry out as concrete hardens. The rate of evaporation being faster than rate of bleeding is typical during placement in hot weather conditions. The surface distress observed in the cores was minor to severe mortar flaking and minor to severe scaling. The mortar flaking was where the paste loss was observed over near surface coarse aggregate particles. The scaling was observed where sub-horizontal micro-cracks lead to loss of material by flaking. Sheet scaling or delamination and popouts were not observed. It was summarized that the cores from Mix 4 and 5, which were not air entrained, exhibited more surface deterioration than cores from Mix 1, 2, and 3.

## 6.2 Examination of Core Body

The in-situ properties of each concrete panel were determined by examination of the body of extracted core by WJE. The examination of core body for quality of materials, structural integrity of concrete and conformance with mix design formulations is essential in determining durability of concrete. The water-cementitious materials ratio, supplementary cementitious material content, paste content, and air void system parameters for all extracted cores are summarized in Table 6.3 through Table 6.9 and are compared to the criteria. The datasheets related to each core are available in WJE report which is attached in the Appendix. Tables 6.2 through 6.7 consists of the results from petrographic analysis of cores extracted from the sidewalk panels. It can be observed from Tables 6.5, 6.6 and 6.7 that the air void system parameters of Mix 4 and 5 are not in conformance with ACI 201.2R recommendations. The air content is very low which is the result of no air entrainment in Mix 4 and 5.

**Table 6.2 Water-cementitious materials ratio results—petrographic analysis**

| Panel Group | Deicing Method | Curing Method | Mix # 1 0.41* | Mix #2 0.43* | Mix #3A 0.42* | Mix #3B 0.44* | Mix # 4 0.51* | Mix #5 0.35* |
|---|---|---|---|---|---|---|---|---|
| A | NaCl | ASTM C1315 Curing and Sealing Compound | 0.35 | - | - | 0.37 | 0.43 | - |
| B | NaCl | Saturated Cover | - | 0.38 | - | - | 0.42 | - |
| B | NaCl | Colloidal Silica Sealer | - | - | - | - | - | - |
| C | NaCl | No Curing | 0.36 | 0.37 | 0.37 | - | 0.42 | - |
| C | NaCl | No Curing | 0.36 | 0.37 | 0.37 | -- | 0.42 | - |
| D | $MgCl_2$ | ASTM C1315 Curing and Sealing Compound | - | 0.37 | - | - | - | 0.37 |
| E | $MgCl_2$ | Saturated Cover | - | - | 0.36 | - | - | - |
| E | $MgCl_2$ | Colloidal Silica Sealer | - | - | 0.37 | - | - | - |
| F | $MgCl_2$ | No Curing | - | - | 0.38 | - | - | - |
| G | Blended Brine (85% NaCl + 15% $MgCl_2$) | ASTM C1315 Curing and Sealing Compound | 0.35 | - | 0.37 | - | - | - |
| H | Blended Brine (85% NaCl + 15% $MgCl_2$) | Saturated Cover | 0.35 | - | - | - | - | 0.37 |
| H | Blended Brine (85% NaCl + 15% $MgCl_2$) | Colloidal Silica Sealer | - | - | - | - | - | - |
| I | Blended Brine (85% NaCl + 15% $MgCl_2$) | No Curing | - | - | - | - | - | 0.37 |

*Note: ACI 201.2R w/cm ratio. Criteria: ≤0.45

As observed in Table 6.2, the w/cm ratio determined for each core is lower than the design w/cm from batch tickets except for Mix 5. It should be noted that the w/cm ratio of all cores, determined from the petrographic analysis, is lower than the ACI 201.2R recommendation of 0.45.

**Table 6.3 SCM content—petrographic analysis**

| Panel Group | Deicing Method | Curing Method | Mix # 1 Fly Ash 25%* | Mix # 2 Slag 49.8%* | Mix # 3A No SCM | Mix # 3B No SCM | Mix #4 Fly Ash 25%* | Mix #5 Slag 50.2 %* |
|---|---|---|---|---|---|---|---|---|
| A | NaCl | ASTM C1315 Curing and Sealing Compound | 25 | - | - | 0 | 25 | - |
| B | NaCl | Saturated Cover | - | 45 | - | - | 25 | - |
| B | NaCl | Colloidal Silica Sealer | - | - | - | - | - | - |
| C | NaCl | No Curing | 25 | 45 | 0 | - | 25 | - |
| C | NaCl | No Curing | 25 | 45 | 0 | - | 25 | - |
| D | $MgCl_2$ | ASTM C1315 Curing and Sealing Compound | - | 45 | - | - | - | 40 |
| E | $MgCl_2$ | Saturated Cover | - | - | 0 | - | - | - |
| E | $MgCl_2$ | Colloidal Silica Sealer | - | - | 0 | - | - | - |
| F | $MgCl_2$ | No Curing | - | - | 0 | - | - | - |
| G | Blended Brine (85% NaCl + 15% $MgCl_2$) | ASTM C1315 Curing and Sealing Compound | 25 | - | 0 | - | - | - |
| H | Blended Brine (85% NaCl + 15% $MgCl_2$) | Saturated Cover | 25 | - | - | - | - | 40 |
| H | Blended Brine (85% NaCl + 15% $MgCl_2$) | Colloidal Silica Sealer | - | - | - | - | - | - |
| I | Blended Brine (85% NaCl + 15% $MgCl_2$) | No Curing | - | - | - | - | - | 40 |

*Note: ACI202.2R SCM Content (%). Criteria: ≤25 fly ash, ≤50 slag

From Table 6.3 it can be observed that the amount of slag is less than that from the actual mix design.

**Table 6.4 Paste content—petrographic analysis**

| Panel Group | Deicing Method | Curing Method | Paste* % Mix 1 | Paste* % Mix 2 | Paste* % Mix 3A | Paste* % Mix 3B | Paste* % Mix 4 | Paste* % Mix 5 |
|---|---|---|---|---|---|---|---|---|
| A | NaCl | ASTM C1315 Curing and Sealing Compound | 32.1 | - | - | 31.0 | 33.9 | - |
| B | NaCl | Saturated Cover | - | 34.7 | - | - | 33.1 | - |
| B | NaCl | Colloidal Silica Sealer | - | - | - | - | - | - |
| C | NaCl | No Curing | 30.9 | 33.6 | 31.7 | - | 34.3 | - |
| C | NaCl | No Curing | 33.6 | 31.1 | 31.7 | | 34.3 | |
| D | $MgCl_2$ | ASTM C1315 Curing and Sealing Compound | - | 32 | - | - | - | 36.6 |
| E | $MgCl_2$ | Saturated Cover | - | - | 31.5 | - | - | - |
| E | $MgCl_2$ | Colloidal Silica Sealer | - | - | 29.5 | - | - | - |
| F | $MgCl_2$ | No Curing | - | - | 33.6 | - | - | - |
| G | B Blended Brine (85% NaCl + 15% $MgCl_2$) | ASTM C1315 Curing and Sealing Compound | 30.2 | - | 31.3 | - | - | - |
| H | Blended Brine (85% NaCl + 15% $MgCl_2$) | Saturated Cover | 31.7 | - | - | - | - | 40.2 |
| H | Blended Brine (85% NaCl + 15% $MgCl_2$) | Colloidal Silica Sealer | - | - | - | - | - | - |
| I | Blended Brine (85% NaCl + 15% $MgCl_2$) | No Curing | - | - | - | - | - | 34.2 |

*AASHT O PP84 Paste Content (%) Criteria: ≤28.0

**Table 6.5 Air content—petrographic analysis**

| Panel Group | Deicing Method | Curing Method | Air content* (%) Mix # 1 | Air content* (%) Mix # 2 | Air content* (%) Mix # 3A | Air content* (%) Mix # 3B | Air content* (%) Mix # 4 | Air content* (%) Mix # 5 |
|---|---|---|---|---|---|---|---|---|
| A | NaCl | ASTM C1315 Curing and Sealing | 6.1 | - | - | 6.8 | 4.8 | - |
| B | NaCl | Saturated Cover | - | 6.1 | - | - | 3.8 | - |
| B | NaCl | Colloidal Silica Sealer | - | - | - | - | - | - |
| C | NaCl | No Curing | 6.8 | 7.8 | 5.9 | - | 4.8 | - |
| C | NaCl | No Curing | 6.4 | 9.8 | 5.9 | - | 4.8 | |
| D | $MgCl_2$ | ASTM C1315 Curing and Sealing | - | 5.8 | - | - | - | 3.6 |
| E | $MgCl_2$ | Saturated Cover | - | - | 7.9 | - | - | - |
| E | $MgCl_2$ | Colloidal Silica Sealer | - | - | 6.8 | - | - | - |
| F | $MgCl_2$ | No Curing | - | - | 6.5 | - | - | - |
| G | Blended Brine (85% NaCl + 15% $MgCl_2$) | ASTM C1315 Curing and Sealing Compound | 6.9 | - | 8.1 | - | - | - |
| H | Blended Brine (85% NaCl + 15% $MgCl_2$) | Saturated Cover | 7.0 | - | - | - | - | 2.0 |
| H | Blended Brine (85% NaCl + 15% $MgCl_2$) | Colloidal Silica Sealer | - | - | - | - | - | - |
| I | Blended Brine (85% NaCl + 15% $MgCl_2$) | No Curing | - | - | - | - | - | 2.9 |

*ACI201.2R Air Content (%) Criteria: 7.0 ± 1.5

**Table 6.6 Air void system spacing factor—petrographic analysis**

| Panel Group | Deicing Method | Curing Method | Air void spacing factor* (in.) Mix #1 | Air void spacing factor* (in.) Mix #2 | Air void spacing factor* (in.) Mix #3A | Air void spacing factor* (in.) Mix #3B | Air void spacing factor* (in.) Mix #4 | Air void spacing factor* (in.) Mix #5 |
|---|---|---|---|---|---|---|---|---|
| A | NaCl | ASTM C1315 Curing and Sealing Compound | 0.006 | - | - | 0.007 | 0.030 | - |
| B | NaCl | Saturated Cover | - | 0.006 | - | - | 0.035 | - |
| B | NaCl | Colloidal Silica Sealer | - | - | - | - | - | - |
| C | NaCl | No Curing | 0.007 | 0.007 | 0.006 | - | 0.034 | - |
| C | NaCl | No Curing | 0.006 | 0.006 | 0.006 | | 0.034 | |
| D | $MgCl_2$ | ASTM C1315 Curing and Sealing Compound | - | 0.005 | - | - | - | 0.067 |
| E | $MgCl_2$ | Saturated Cover | - | - | 0.005 | - | - | - |
| E | $MgCl_2$ | Colloidal Silica Sealer | - | - | 0.012 | - | - | - |
| F | $MgCl_2$ | No Curing | - | - | 0.006 | - | - | - |
| G | Blended Brine (85% NaCl + 15% $MgCl_2$) | ASTM C1315 Curing and Sealing Compound | 0.006 | - | 0.006 | - | - | - |
| H | Blended Brine (85% NaCl + 15% $MgCl_2$) | Saturated Cover | 0.005 | - | - | - | - | 0.064 |
| H | Blended Brine (85% NaCl + 15% $MgCl_2$) | Colloidal Silica Sealer | - | - | - | - | - | - |
| I | Blended Brine (85% NaCl + 15% $MgCl_2$) | No Curing | - | - | - | - | - | 0.088 |

* Air void system spacing factor (in.) ACI 201.0R Criteria: ≤ 0.008

**Table 6.7 Air void system specific surface area—petrographic analysis**

| Panel Group | Deicing Method | Curing Method | Air void surface area* ($in^2/in^3$) Mix # 1 | Air void surface area* ($in^2/in^3$) Mix # 2 | Air void surface area* ($in^2/in^3$) Mix # 3A | Air void surface area* ($in^2/in^3$) Mix # 3B | Air void surface area* ($in^2/in^3$) Mix # 4 | Air void surface area* ($in^2/in^3$) Mix # 5 |
|---|---|---|---|---|---|---|---|---|
| A | NaCl | ASTM C1315 Curing and Sealing Compound | 795 | - | - | 617 | 181 | - |
| B | NaCl | Saturated Cover | - | 775 | - | - | 169 | - |
| B | NaCl | Colloidal Silica Sealer | - | - | - | - | - | - |
| C | NaCl | No Curing | 678 | 595 | 857 | - | 159 | - |
| C | NaCl | | 769 | 511 | 857 | - | 159 | - |
| D | MgCl2 | ASTM C1315 Curing and Sealing Compound | - | 893 | - | - | - | 95 |
| E | $MgCl_2$ | Saturated Cover | - | - | 753 | - | - | - |
| E | $MgCl_2$ | Colloidal Silica Sealer | - | - | 347 | - | - | - |
| F | $MgCl_2$ | No Curing | - | - | 797 | - | - | - |
| G | Blended Brine (85% NaCl + 15% $MgCl_2$) | ASTM C1315 Curing and Sealing Compound | 728 | - | 668 | - | - | - |
| H | Blended Brine (85% NaCl + 15% $MgCl_2$) | Saturated Cover | 828 | - | - | - | - | 134 |
| H | Blended Brine (85% NaCl + 15% $MgCl_2$) | Colloidal Silica Sealer | - | - | - | - | - | - |
| I | Blended Brine (85% NaCl + 15% $MgCl_2$) | No Curing | - | - | - | - | - | 77 |

*Air void system specific surface area ($in^2/in^3$) ACI201.0R Criteria: ≥600

The following comments in this section are summaries of observations and results reported in WJE report *Laboratory Studies of Twenty-One Concrete Cores* (Results and observations are available in full detail in WJE report attached in Appendix).

### 6.2.1 Aggregates

Many of the coarse aggregates in the cores were composed of crushed siliceous igneous and meta-igneous rocks. The coarse aggregate color ranged from light to dark gray. The maximum nominal top size of coarse aggregate observed ranged from ½ to ¾ in. The coarse aggregates were angular to subangular and blocky to elongated/flat. The elongated/flat particles showed a sub-parallel orientation near surface in few cores.

Siliceous sand fine aggregate contains quartz/quartzite and feldspar, and small amounts of micas, amphibole, schist, microcrystalline quartz, and ironstone. Along with fine aggregates, sand-size particles of coarse aggregate were observed in cores. The quartz mineral observed in fine and coarse aggregates tends to react with alkalis from portland cement paste (ASR reaction). No evidence related to ASR reaction was observed in the cores. The supplementary cementitious materials used in the mix design help mitigate possibility of ASR reaction. The staining observed on surface of 2D-1 core was a result of iron sulfide mineralization in near surface coarse aggregate particles, it does not affect the durability of concrete.

All cores exhibited moderately weak to moderately strong paste-aggregate bonds. The cores containing fly ash showed moderately weak to moderately strong bond strength and core containing slag showed moderately strong to strong bond strength. While Mix 3, which had no SCM exhibited moderately strong bond strength.

### 6.2.2 Paste

The paste below the near surface of cores containing slag cement exhibited a blue-green tint, which is usually observed in concrete with slag cement that had not been exposed to air. With increasing time of air exposure in laboratory, the blue-green color faded to varying degrees. The cores containing fly ash or no SCM exhibited a light brownish gray color. However, a few cores had a slightly darker layer of paste along the top surface corresponding with depth of carbonation.

The paste had a hardness close to 3 on the Mohs hardness scale for hard paste. While no residue was left from scratching copper probe on the moderately hard paste. The presence of calcium hydroxide crystals was low, which is typical for concrete containing SCMs and low to moderately low w/cm ratios. The observed calcium hydroxide crystals ranged from tabular to poorly defines in shape. As noted in the WJE report, “the hydration of portland cement appeared to be normal to advanced overall.”

### 6.2.3 Air-Void System

Mix 1, 2, and 3 were intentionally air entrained, while Mix 4 and 5 were not air entrained. The air void analysis of Mix 1,2, and 3 cores meets with the minimum ACI 201.2R recommendations. The air content was within 7 ± 1.5% and the spacing factor was less than

0.008 in. However, the specific surface for a few cores (Cores 2C-1, 2C-2, and 3E-CSS-1) are marginally low compared to the ACI recommendation of 600 $in^2/in^3$. As noted in WJE report, “the specific surface is a non-additive average parameter and can be biased lower that the ACI recommended 600 $in^2/in^3$ if the concrete contains large entrapper air voids, even if the small air voids alone in the concrete are sufficient to meet the requirements”. The concrete cores of Mix 4 and 5 which were intentionally not air entrained, did not exhibit the air void system standards for protection against freezing and thawing cycles. The entrained air voids were uniformly distributed in the concrete cores of Mix 1, 2, and 3. While, the cores from Mix 5 had high volume of large irregularly shaped consolidated voids.

### 6.2.4 Cracking

Within the near surface layer of the cores, surface-perpendicular and surface-parallel micro-scale cracks were observed in most of the cores. However, no large-scale cracks were observed. The perpendicular micro-cracks are typically a result of drying shrinkage. While parallel micro-cracks are due to cyclic freezing and thawing. Very short microcracks which are visible during thin-section studies, were also frequently observed in the body of concrete cores with low w/cm ratio.

### 6.2.5 Carbonation

The depth of the carbonation was generally within the top 0.08 to 0.3 in. of concrete and varied among the cores. The depth of carbonation is slightly deeper where near surface microcracks and voids are present.

### 6.2.6 Secondary Deposits

As reported in WJE report, “Secondary deposits of ettringite were observed to partially to fully line a majority of air voids in the Mix 1 cores. Secondary deposits of ettringite and/or calcium hydroxide were observed in a minor proportion of air voids in the remaining cores. Air voids were generally free of secondary deposits in Mix 2 cores”.

### 6.2.7 Concrete Mixture Compliance

From the petrographic studies, it was observed that the cores generally were well consolidated with varying amounts of entrapped and consolidation voids. A weak layer near surface was not observed in the cores and the distribution of aggregate, paste, and air-voids was generally uniform in the body. Comparing the petrographic results and corresponding mix designs, the concrete was generally consistent with the mix design except for total aggregate content and paste content. The total aggregate content was lower than the design value and the paste content was higher than the design value in all cores.

Mix 1 (5 Cores)

The mix design value of total aggregate content was 36.63%, while the measured total aggregate volume ranged from 35.1 to 37.7%. The design value of paste volume was lower than measured paste volumes. For core 1C-2, the measured paste volume was 8.4% higher than design value. The measured hardened air contents for all Mix 1 cores were below the estimated air content of mix design (9.7%). The estimated fly ash contents ranged from 20 to 30%, which closely matched with mix design value of 25%.

Mix 2 (4 Cores)
The measured total aggregate volumes were less than design value of aggregate content and the measured paste volumes were higher than design value of paste volumes in all cores of Mix 2. The measured hardened air content varied from 5.8 to 9.8%. The estimated air content from the mix design was within the range (8.3%). The estimated slag contents were in the range of 40 to 50%.

Mix 3 (6 Cores)
The measured total aggregate volumes were always lower than the design value in cores of Mix 3. The coarse aggregate content varied up to 2.2% from the design value whereas the fine aggregate content varied up to 4.8% for Core 3C-1. The measured paste volumes were greater than design paste content of the mix. The hardened air contents range from 5.9 to 7.9% which was lower than estimated design air content. The cores of Mix 3 did not contain any SCMs, complying with mix design which contained only portland cement.

Mix 4 (3 Cores)
The measured total aggregate volume was less than the design value and measured paste volumes were greater than design value. For core 4C-1, the measure total aggregate volume was 7.5% less than as-designed and the paste volume was 6.5% higher than as-designed. Even though the Mix 4 was not air entrained, air content ranging from 3.8 to 4.8% were measured in the core. The as-designed air content of Mix 4 was 3.7%. The estimated fly ash contents were between 20 to 30% complying with the 25% replacement in mis design.

Mix 5 (3 Cores)
The measured total aggregated volumes were less than the design value of Mix 5. The measured paste volumes were higher than the design value. Mix 5 was intentionally not air entrained, but air contents ranging from 2.0 to 3.6% were measured. The air content estimated for the mix design was 3.2%. The estimated slag content ranges from 35 to 45% which is less than as-designed value of 50% replacement in the mix. WJE report noted that modern slag cement is extremely fine making it difficult to estimate slag cement contents in thin sections.

# 7.0 Photogrammetric Analysis

The concrete sidewalks were regularly photographed from November 2021 to April 2022. These panels were used to compute the percent scaling of the sidewalks. The pictures were taken using a Canon EOS Rebel T6 DSLR and the photo rig setup, as shown in Figure 7.1. The three sides of the canopy setup were covered to create approximately the same lighting for all the sets of pictures documented. Table 7.1 consists of the date of photographs taken.

**Table 7.1 Dates of panel documentation**

| Dates of Photographs |
|---|
| November 18, 2021 |
| December 20, 2021 |
| January 4, 2022 |
| January 16, 2022 |
| February 9, 2022 |
| March 4, 2022 |
| March 30, 2022 |
| April 25, 2022 |

**Figure 7.1 Photo rig setup to document sidewalk panels**

## 7.1 Visual Examination

The first set of documented photographs taken before the first seasonal freeze was compared to photographs taken on April 25 to analyze the total deterioration after a complete winter season. A visual analysis was done to identify a rough estimate of the percentage of the panel that is scaled. Based on the ASTM C672 rating, the types and severity of scaling can be divided into a few categories:

Low—superficial damage where no coarse aggregates are showing
Medium—some coarse aggregates are showing, and a small amount of surface area is lost
High—severe scaling with a substantial amount of coarse aggregate showing and a significant amount of surface area is lost

The panels cured with a saturated cover had brown and black burlap stains due to the use of unclean burlap. These stains did not fade away with time. The ASTM C1315 Type I curing and sealing compound had a red fugitive dye which left pink stains on the panels. However, the pink stains gradually faded as the concrete matured, though at the conclusion of the study, application marks from the dye were still visible.

The Mix 3 panels showed no signs of scaling. All panels of Mix 4 except 4A have shown scaling. The most severe scaling has been observed in Mix 5 and Mix 4 with no curing panels. For Mix 5, no curing panels and coarse aggregates are visible. However, for Mix 4 no curing panels, only loss of mortar is observed, and there is no visibility of coarse aggregates. Few panels of Mix 1 and Mix 2 have shown minor scaling.

Table 7.2 consists of human visual examination results for all 48 panels. The human visual scaled area percentage is average of four engineers estimating the percentage of surface area scaled out of 100%. The engineers were not allowed to confer with each other, and were shown the same sidewalk image simultaneously. An average percent scaling has also been identified for deciding groups, curing groups, and mix design.

**Table 7.2 Visual examination results**

| Panel Group | Deicing Method | Curing Method | Percent Scaled Mix # 1 | Percent Scaled Mix # 2 | Percent Scaled Mix # 3A | Percent Scaled Mix # 3B | Percent Scaled Mix # 4 | Percent Scaled Mix # 5 |
|---|---|---|---|---|---|---|---|---|
| A | NaCl | ASTM C1315 Curing and Sealing | 0 | 7 | - | 2 | 1 | 3 |
| B | NaCl | Saturated Cover | 0 | 0 | 1 | - | 21 | 1 |
| B | NaCl | Colloidal Silica Sealer | - | - | 0 | 0 | - | - |
| C | NaCl | No Curing | 4 | 20 | 1 | - | 18 | 19 |
| D | $MgCl_2$ | ASTM C1315 Curing and Sealing | 1 | 4 | 0 | - | 29 | 1 |
| E | $MgCl_2$ | Saturated Cover | 0 | 0 | 1 | - | 2 | 0 |
| E | $MgCl_2$ | Colloidal Silica Sealer | - | - | 1 | - | - | - |
| F | $MgCl_2$ | No Curing | 2 | 1 | 1 | - | 14 | 33 |
| G | Blended Brine (85% NaCl + 15% $MgCl_2$) | ASTM C1315 Curing and Sealing Compound | 2 | 8 | 0 | - | 3 | 2 |
| H | Blended Brine (85% NaCl + 15% $MgCl_2$) | Saturated Cover | 1 | 1 | 0 | - | 7 | 0 |
| H | Blended Brine (85% NaCl + 15% $MgCl_2$) | Colloidal Silica Sealer | - | - | 0 | - | - | - |
| I | Blended Brine (85% NaCl + 15% $MgCl_2$) | No Curing | 2 | 1 | 1 | - | 19 | 15 |

## 7.2 Computer Based Photogrammetric Examination

Photogrammetric analysis was performed using the Image Processing extension in MATLAB. The photos of sidewalk panel documents are cropped and corrected using a Key Stone Corrector application available on MATLAB. The photogrammetric analysis code from (Phase -1 citation) has been adopted. Few changes were made to the program for this study.

The percentage of scaling is determined by categorizing the grey color on the image. The photograph is converted to a greyscale image such that each pixel in the image corresponds to a color on a scale of grey from 0 – 1. All the photos of the panels are corrected using a code created by UMass Amherst Professor Dr. Chengbo Ai. This code corrects the greyscale image if it has any heavy shadows or stains. For selecting the threshold to compute the percentage of scaling of a panel, histograms of pixels were created for each grayscale panel image as shown in Figure 7.2. The histogram is essentially the occurrences of each color of the greyscale. For these histograms, mean and standard deviations were calculated. A standard deviation is chosen as the threshold to calculate percent scaling. The threshold is used for determining which pixels in a gray-scaled image are considered scaled. The threshold varies for each panel depending on color of panel, color of scaled portion, finishing markings and curing stains. Then the photos are processed through the MATLAB code using the selected thresholds to get the percent scaling for each panel.

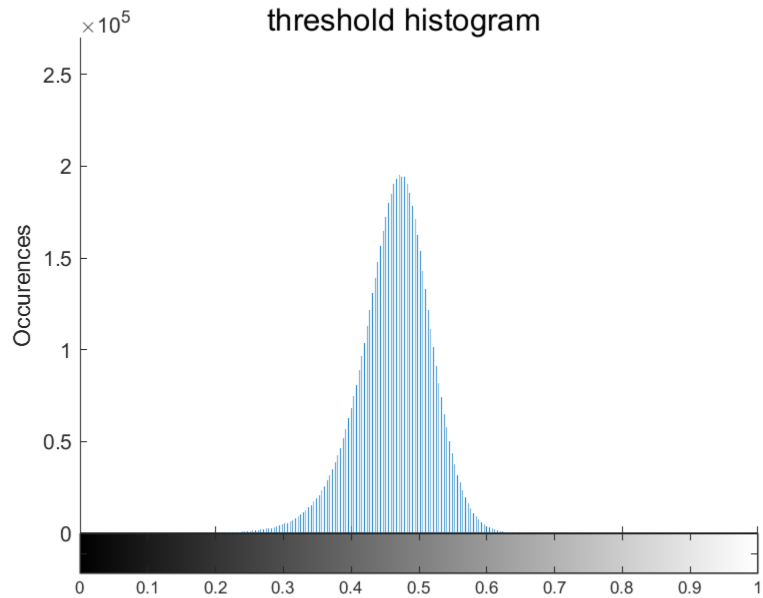


**Figure 7.2 Histogram of panel 1D**

For all panels which had moderate to severe scaling, CDF plots were created. The probability of the selected threshold for the panels is plotted on the CDF graph. The CDF plot was created to investigate whether a relation exists between all the thresholds selected for panels as the threshold for each panel is different. From Figure 7.3 it can be observed that the thresholds selected for computing percent scaling are at 0% percentile. Also, the shape of the CDF curve depends of the spread of pixel colors, which changes based on the curing method.

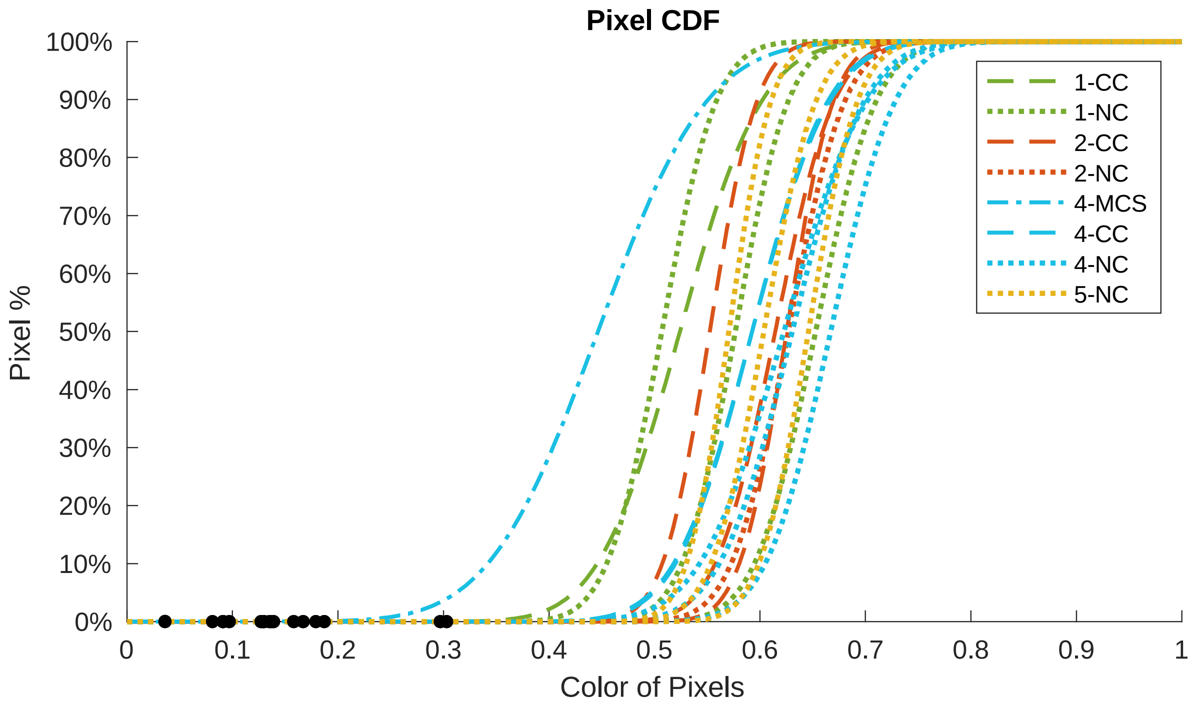


**Figure 7.3 CDF plots of the scaled sidewalk panels**

In the case of saturated cover curing, the dark burlap stain on the panels made it difficult to compute percent scaling. In the few saturated covers curing panels where scaling is observed, the percent scaling is subtracted from 100% to compute the actual scaling, which is much lighter in color than burlap stains. In several panels, stains and broom finishing strokes were identified as scaling, resulting in a higher percentage of scaling. To eliminate this overestimation, a percent scaling is computed for photos of panels taken before deterioration begins and then subtracted from the panel with scaling. This procedure produces accurate scaling by eliminating finishing marks and stains. However, there is a limitation to the procedure. As concrete panels mature, some of the stains which existed initially have faded over time. Shadows, lighting, dirt, watermarks, curing stains, and discoloration affect the accuracy of this analysis method.

Table 7.3 consists of percent scaling computed for all 48 panels using computer-based photogrammetric analysis. From comparing the percent scaling determined from photogrammetric analysis to visual examination, it can be observed that human visual examination overestimates the percentage of the scaled area on the panel in most cases. For example, panels 4D and 4C which have a significant amount of scaling. On visual examination, it is determined that 4C has 18% and 4D has 29% scaled area. However, from photogrammetric analysis, it is determined that 4C is more scaled than 4D with a difference of 2.8%. Similarly, for panel 3A, the percentage of scaled area was 2% from visual examination. However, no signs of scaling were observed which was detected in the photogrammetric analysis. During visual examination, it is observed that the stains and other marks on the surface are typically counted toward percent scaling. Also, from person to person, the way of analyzing the percent scaling is changing. These differences in human and computer-based analysis give a stronger foundation for the development of programs to determine percentage of scaling with more accuracy.

**Table 7.3 Computer-based scaling percentage from photogrammetric analysis**

| Panel Group | Deicing Method | Curing Method | Percent Scaled Mix #1 | Percent Scaled Mix #2 | Percent Scaled Mix #3A | Percent Scaled Mix #3B | Percent Scaled Mix #4 | Percent Scaled Mix #5 |
|---|---|---|---|---|---|---|---|---|
| A | NaCl | ASTM C1315 Curing and Sealing Compound | 0.0 | 2.8 | - | 0.0 | 0.0 | 0.0 |
| B | NaCl | Saturated Cover | 0.0 | 0.0 | 0.0 | - | 8.8 | 0.0 |
| B | NaCl | Colloidal Silica Sealer | - | - | - | 0.0 | - | - |
| C | NaCl | No Curing | 0.1 | 5.4 | 0.0 | - | 9.6 | 2.2 |
| D | $MgCl_2$ | ASTM C1315 Curing and Sealing Compound | 0.0 | 0.6 | 0.0 | - | 6.8 | 0.0 |
| E | $MgCl_2$ | Saturated Cover | 0.0 | 0.0 | 0.0 | - | 0.0 [1] | 0.0 |
| E | $MgCl_2$ | Colloidal Silica Sealer | - | - | 0.0 | - | - | - |
| F | $MgCl_2$ | No Curing | 0.1 | 0.1 | 0.0 | - | 2.2 | 2.4 |
| G | Blended Brine (85% NaCl + 15% $MgCl_2$)) | ASTM C1315 Curing and Sealing Compound | 0.1 | 1.1 | 0.0 | - | 1.2 | 0.0 |
| H | Blended Brine (85% NaCl + 15% $MgCl_2$) | Saturated Cover | 0.0 [1] | 0.0 [1] | 0.0 | - | 0.0 [1] | 0.0 [1] |
| H | Blended Brine (85% NaCl + 15% $MgCl_2$) | Colloidal Silica Sealer | - | - | 0.0 | - | - | - |
| I | Blended Brine (85% NaCl + 15% $MgCl_2$) | No Curing | 0.7 | 0.0 | 0.0 | - | 6.1 | 2.2 |

Notes:

[1] The photogrammetric program was unable to find percent scaling on these moisture-cured panels. The scaling on these panels was only a few flakes. Therefore, the percent scaling is considered as 0%.

# 8.0 Analysis of Results

## 8.1 Mix Design Formulation

Table 8.1 provides a summary of all criteria used to assess compliance of all concrete mix design formulations. The criteria identified in Table 8.1, when met, will result in increased resistance to freezing, thawing, and deicing damage. However, all the mix design formulations do not fully meet the criteria. Mix 1 and 3 have the highest conformance out of all other mixes. Both Mix 4 and 5 have low conformance.

**Table 8.1 Mix design formulation analysis[1]**

| Property | Property | Mix 1 | Mix 2 | Mix 3 | Mix 4 | Mix 5 | Criteria[2] |
|---|---|---|---|---|---|---|---|
| Combined Aggregate Gradation | Tarantula Curve | Pass | Pass | Pass | Fail | Fail | Pass |
| Combined Aggregate Gradation | Shilstone Workability-Coarseness Chart | Zone II | Zone II | Zone II | Zone IV | Zone II | Zone II |
| Paste System | Freezing, Thawing, and Deicing Resistance, w/cm | 0.41 | 0.43 | 0.42 | 0.51 | 0.35 | ≤ 0.45 |
| Paste System | SCM Content (%)Fly Ash | 25 | - | - | 25 | - | ≤ 25.0 |
| Paste System | SCM Content (%)Slag | - | 49.8 | - | - | 50.2 | ≤ 50.0 |
| Paste System | Water-Reducing Admixtures | Yes | Yes | Yes | Yes | Yes | Yes |
| Paste System | Paste Content (%) | 29.0 | 28.7 | 28.1 | 30.7 | 32.8 | ≤ 28.0 |
| Paste System | Paste Content to Void Content Ratio (PC/VC) | 1.2 | 1.2 | 1.2 | 1.2 | 1.3 | 1.25–1.75 |
| Air Void System | Air Content (%)[3] | 6.2 | 5 | 6.2 | 3.1 | 1.5 | 7.0 ± 1.5 |
| Air Void System | Air-Entraining Admixtures | Yes | Yes | Yes | No | No | Yes |
| - | Conformance (%) | 77.8 (7/9) | 77.8 (7/9) | 75.0 (6/8) | 22.2 (2/9) | 44.4 (4/9) | 100 (9/9) |

Notes:

[1] Cells highlighted in orange color have exceeded the given criteria.

[2] The criteria recommended by *Integrated Materials and Construction Practices for Concrete Pavement: A State-of-the-Practice Manual (13)*

[3] The air content for the mix design formulations were not provided by the cement concrete producer. Therefore, the fresh concrete air content results from AASTHO T 152 Air Content of Freshly Mixed Concrete by the Pressure Method were reported in this table.

## 8.2 Field and Laboratory Testing

The fluid and hardened concrete testing criteria identified in Table 8.2, when met, will result in increased resistance to freezing, thawing, and deicing damage. All the mix design formulations, however, did not fully meet the criteria. The highest conformance based on the given criteria given was observed for Mix 3. Mix 4 had the lowest conformance in both mix design formulation and concrete properties. The lowest conformance of Mix 4 and 5 for both mix design and concrete analysis is expected as these concrete mixes were intentionally designed to violate aggregate and paste contents, so they are considered to be poor mixes.

**Table 8.2 Fresh and hardened concrete analysis[1]**

| Test Method | Property | Mix 1 | Mix 2 | Mix 3[2] | Mix 4 | Mix 5 | Criteria[3] |
|---|---|---|---|---|---|---|---|
| T 152 | Air Content (%) | 6.2 | 5 | 6.2 | 3.1 | 1.5 | 7.0 ± 1.5 |
| T 309 | Concrete Temperature (F) | 82 | 88 | 88 | 86 | 90 | 60-90 |
| T 22 | Compressive Strength (psi) 7 days | - | 3080 | 3650 | 2320 | - | ≥ 2800 |
| T 22 | Compressive Strength (psi) 28 days | 3840 | 4890 | 4640 | 3270 | 5380 | ≥ 4000 |
| T 22 | Compressive Strength (psi) 91 days | 4910 | 5750 | 5130 | 4250 | 6330 | ≥ 4000 |
| T 358 | Resistivity (kΩ-cm) 28 days | 11.4 | 17.2 | 8.9 | 6.6 | 30.8 | ≥ 21.0 |
| T 358 | Resistivity (kΩ-cm) 91 days | 21.5 | 32.2 | 10.9 | 13.5 | 34.4 | ≥ 21.0 |
| C 672 | Scaling Resistance: Standard Moist Cure (Rating) | 2.5 | 5 | 1.5 | 5 | 5 | ≤ 2.0 |
| NQ 2621-900 | Scaling Resistance: Curing using Saturated Cover ($kg/m^2$) | 0.03 | 0.04 | 0.005 | 1.21 | 0.18 | < 0.5 |
| NQ 2621-900 | Scaling Resistance: Curing using Sealing and Curing Compound ($kg/m^2$) | 0.45 | 1.33 | 0.45 | 3.50 | 0.98 | < 0.5 |
| NQ 2621-900 | Scaling Resistance: Curing using Colloidal Silica Sealer ($kg/m^2$) | - | - | 0.25 | - | - | < 0.5 |
| NQ 2621-900 | Scaling Resistance: No Curing ($kg/m^2$) | 0.22 | 0.88 | 0.075 | 4.56 | 2.21 | < 0.5 |
| TP 119 | Uniaxial Resistivity (kΩ-cm) 28 days | 5.32 | 9.34 | 4.45 | 3.61 | 11.6 | ≥ 21.0 |
| TP 119 | Uniaxial Resistivity (kΩ-cm) 91 days | 12.1 | 17.6 | 5.98 | 7.33 | 19.4 | ≥ 21.0 |
| C 457[4] | Air Content (%) | 6.64 | 7.37 | 7.00 | 4.46 | 2.83 | 7.0 ± 1.5 |
| C 457[4] | Spacing Factor (in) | 0.006 | 0.006 | 0.007 | 0.033 | 0.073 | ≤ 0.008 |
| C 457[4] | Specific Surface Area ($in^2/in^3$) | 759.6 | 693.5 | 673.2 | 169.6 | 102.0 | ≥ 600 |
| - | Conformance (%)[5] | 66.7 10/15 | 56.2 9/16 | 76.5 13/17 | 12.5 2/16 | 37.5 6/16 | 100% |

Notes:

[1] Cells highlighted in orange color have exceeded the given criteria.

[2] The hardened concrete testing was not conducted for both Mix 3A and 3B. The results in the table are conducted for Mix 3A. It should also be noted that only 2 sidewalk panels were placed with Mix 3B and all the test samples placed during placement are of Mix 3A.

[3] The criteria recommended by *Integrated Materials and Construction Practices for Concrete Pavement: A State-of-the-Practice Manual (13)*

[4] The air void system parameters reported in the table are average of cores examined for each mix design.

[5] The number of passing results varied in the mix designs because for Mix 1 and 5 compressive strengths at 7 days is not available. A colloidal silica sealer was only used for three panels of Mix 3.

### 8.2.1 Air Void System

The air content results from ASTM C457 for each mix in Table 8.2 are average of cores examined for each mix design. The air content was within the ACI 201.2R recommended values except for Mix 4 and 5 which were intentionally not air entrained. The air content estimated from petrographic results is always greater than fresh concrete air content (AASHTO T 152).

### 8.2.2 Batch Material Quantities Versus Material Quantities Estimated from Petrographic Analysis

It was observed that there is a discrepancy between the material quantities from batch tickets during placement and results from petrographic analysis (*ASTM C856 Petrographic Examination of Hardened Concrete*) *(12)*. Both Mix 1 and 4 had the same amount of fly ash in batch tickets and petrographic analysis results (25%). While there was a difference in slag quantities in both Mix 2 and 5. For both the mix design, the amount of slag was lower than the mix design value from batch tickets (~ 50%). The water-cementitious ratio determined from petrographic results were lower than the water-cementitious ratio computed from the batch tickets. For all mix designs, the paste content determined from petrographic results were always greater than design values.

**Table 8.3 Batch ticket versus petrographic hardened concrete cylinders results[1]**

| Property | Metric | Mix 1 | Mix 2 | Mix 3 | Mix 4 | Mix 5 |
|---|---|---|---|---|---|---|
| Slag Content (%) | Design | - | 49.8 | - | - | 50.2 |
| Slag Content (%) | Petrography | - | 45 | - | - | 40 |
| w/cm ratio | Design | 0.41 | 0.43 | 0.43 | 0.51 | 0.35 |
| w/cm ratio | Petrography | 0.35 | 0.37 | 0.37 | 0.42 | 0.37 |
| Paste content (%) | Design | 29.0 | 28.7 | 28.1 | 30.7 | 32.8 |
| Paste content (%) | Petrography | 31.7 | 32.8 | 31.4 | 33.8 | 37.0 |

Notes:

[1] The petrographic study results are average of cores examined for each mix design.

### 8.2.3 Scaling Resistance Test Results

For BNQ NQ 2621-900 (conducted at UMass Amherst), the passing limit for the test is 0.5 kg/m$^2$. Typically, the test ends after 56 cycles but in this study the cycles extended until 112 freeze-thaw cycles. The ASTM C672 was conducted at MassDOT laboratory until 50 freeze-thaw cycles and the specimens were subjected to standard moist curing.

All specimens of Mix 4 and Mix 5 except Mix5 with saturated cover curing failed after 56 freeze-thaw cycles. All of Mix 4 specimens have exceeded the passing limit of the test after 7 freeze-thaw cycles. Mix 5 specimens subjected to curing and sealing compound and no curing have all failed the test after 21 cycles. Mix 5 specimens which were cured using a saturated cover has lasted until 77 freeze-thaw cycles. By end of 77 freeze-thaw cycles, all specimens with Mix 4 and Mix5 have exceeded the 0.5 kg/m$^2$ limit. Mix 4 and Mix5 are the "poor" mix designs which had excess fine aggregates and excess coarse aggregates respectively. Air-entraining admixtures were not utilized in these mix designs. The excessive scaling observed in Mix 4 and Mix5 specimens might be the consequence of poor aggregate gradation and improper air-void system.

Specimens of Mix 2 which were subjected to saturated cover curing had passed the limit of the test after 112 freeze-thaw cycles. However, no curing and curing with chemical compound of Mix 2 specimens results in failure of specimens by 21 freeze-thaw cycles. Curing with saturated cover seems to be beneficial when high replacement of slag is used in the mix (Mix 2 has 50.2% slag). As moisture curing when done right will provide adequate moisture and sufficient time for concrete to gain strength and mature. The reason for failure of Mix 2 specimens might be high replacement of slag.

All specimens of Mix 1 and 3have passed the BNQ NQ 2621-900 limit after 56 freeze-thaw cycles. After 100 freeze-thaw cycles, Mix 1 and Mix 3 subjected to chemical compound curing have failed the test. The better performance of Mix 1 and 3 compared to other mixes might be due to adequate air entrainment, w/cm, proper aggregate gradation and limited (25% Fly ash in Mix 1) or no (Mix 3) SCMs in concrete.

All specimens with chemical compound curing have failed the test by 112 freeze-thaw cycles. The early application of chemical curing compound will entrap bleed water under the membrane which will lead to excess w/cm at top surface and resulting in a weak top surface layer susceptible to scaling.

The visual rating of specimens of BNQ with no curing relate closely to rating of ASTM C672. For Mix 1 with no curing the visual rating was 3 and 2.5 for BNQ and ASTM method after 56 and 50 freeze-thaw cycles respectively. For Mixes 2,4 and 5, the visual rating for both ASTM and BNQ with no curing and chemical compound curing was 5 after 50 and 56 cycles respectively. In case of 3, the rating was 2 and 1.5 for BNQ and ASTM after 56 and 50 cycles respectively.

## 8.3 Petrographic Analysis

The following commentary is the summarization of discussion and recommendations from WJE's *Laboratory Studies of Twenty-One Concrete Cores* report which is available in Appendix.

The petrographic study was conducted on the twenty-one concrete cores extracted from sidewalk panels to identify the cause of deterioration of concrete surface in form of mortar flaking and scaling. The surface distress observed is consistent with scaling which occurs when saturated concrete is subjected to freezing and thawing cycles. Scaling can accelerate when exposed to relatively low concentrations of deicing salts. Various factors that can affect risk of scaling are ponding of water on surface or contact with saturated coil, and inadequate curing during placement. The resistance against surface distress can be increased by proper curing and finishing of concrete along with adequate air entrainment and low w/cm ratio.

Concrete mixes with high SCM content require extended curing duration and are susceptible to near-surface carbonation and scaling.

Most of the surface distress observed in the cores is concluded to be minor to moderate mortar flaking. Mortar flaking can occur when the localized weak surface paste above coarse aggregates near surface are subjected to cyclic freezing, thawing and deicers. Inadequate curing can dry out the concrete surface and lead to mortar flaking. Surface drying is a common occurrence when the surface is not protected from high heat, high winds, and high sun exposure after placement, or when the rate of bleeding is slow. As reported in WJE report, "Mortar flaking and scaling are not mutually exclusive, and can occur together, or deterioration that may begin as mortar flaking may advance to more severe scaling".

The observed mortar flaking in a few cores suggests that a localized weaker layer was created either due to hard-troweling of air-entrained concrete or sub-parallel alignment of blocky or elongated coarse aggregate particles near surface. When the elongated or blocky coarse aggregate particles are aligned sub-parallel, the bleed water is prevented from reaching the surface and hydrate the paste will result in a weak paste layer above the particles. Proper finishing practices and adequate curing are effective methods to reduce the formation of weak layer in concrete. When supplementary cementitious materials are incorporated in concrete, adequate curing and timely finishing practices must be followed to ensure necessary strength gain before exposure to freezing environment and deicing agents. The fine fly ash and slag particles block pores in the concrete which reduces the rate of bleeding in the concrete. For the concrete containing SCMs or has a low w/cm ratio, the timing of finishing practices must be adjusted accordingly. Setting time and rate of strength gain are delayed when SCMs are incorporated in concrete. Therefore, extended curing durations are required for adequate strength gain before exposure to freezing environment and deicing salts. In the twenty-one cores examined, untimely finishing was not observed. However, the evidence of mortar flaking may be a result of inadequate curing. For concrete exhibiting minimal bleeding, moisture curing is more effective than use of curing compounds as moist curing can supply moisture needed for hydration. Therefore, reducing preventing mortar flaking.

From the petrographic studies, it was observed that curing has not be adequate in some cores that received curing compound. The delayed setting time and reduced rate of bleeding in concrete with SCMs generally affect time of supplication of curing compound and effectiveness of curing compound. Curing compounds are not as effective if they are applied after loss of internal moisture in concrete as it does not supply additional moisture for hydration process. Therefore, in the case of concrete which shows minimal bleeding moisture curing methods should be preferred rather than use of curing compounds. However, in the twenty-one cores examined, the severity of mortar flaking is less which suggests that the timing and application of curing compound was successful.

The concrete mixes which were air entrained (Mix 1, 2, and 3) have exhibited air void parameters consistent with ACI 201.2R recommendations for freeze-thaw durability except for a few cores which exhibited specific area lower than the recommended value. From examination of all cores, it was concluded that no direct correlations was established between mix designs and surface deterioration observed. Mortar flaking was observed in cores of each mis design. While scaling was only observed in Core 2C-1 and 4B-1. Even though Mix 4 and 5 are not entrained, they containing a small amount of air voids which may have aided in freezing and thawing durability. The concrete cores which had localized weak top layers are expected to perform better after the weak paste is eroded. The air voids in the concrete body of Mix 1,3 and 3 are small and evenly distributed which protects the concrete from deterioration due to freezing and thawing cycles. Mixes 4 and 5 had showed the higher overall surface distress among all mixes examines. The Mix 5 which has lower design w/cm is expected to perform better than Mix 4 because a low w/cm ratio will result in higher relative strength of paste and lower permeability in the concrete. The cores which were not air entrained exhibited more distress than cores with air entrainment. However, the effect of air entrainment will be more evident in future after cyclic exposure to freeze-thaw cycles and deicing agents. It was observed that most of the cores showed almost negligible chloride ion concentrations on surface which indicated that exposure to deicing salts was not as aggressive as expected in new concrete placed before winters.

## 8.4 Photogrammetric Analysis

The results from photogrammetric analysis are summarized based on curing method, deicing method and mix design in this section. The average percentage of scaled area of panels are computed from the results tabulated in Table 7.3. From Table 8.4, it can be observed that the panels which were not subjected to curing have shown a higher scaling percentage. The panels which were subjected to moisture curing have shown slightly better performance than the ones cured using chemical compounds.

**Table 8.4 Computer-based photogrammetry results by curing method**

| Ranking | Curing Method | Average Percentage of Scaled Area (%) |
|---|---|---|
| 1 | Moisture curing using a wet burlap | 0.6 |
| 2 | Use of ASTM C1315 Curing and Sealing Compound | 0.8 |
| 3 | No curing | 2.1 |

Table 8.5 shows the average percentage of scaled area measured based on deicing methods. Panels exposed to blended brine have shown the highest resistance to scaling. The panels subjected to 30% by wt. magnesium chloride ($MgCl_2$) solution have shown slightly lower resistance compared to the ones exposed to blended brine. Panels subjected to sodium chloride have scaled more than 2 times than the ones exposed to $MgCl_2$ solution. It should be noted that until January 2022, the blended brine solution was not utilized. Therefore, during the initial freezing and thawing cycles, the panels were not in contact with any deicing agents.

**Table 8.5 Computer-based photogrammetry results by deicing method**

| Ranking | Deicing Method | Average Percentage of Scaled Area (%) |
|---|---|---|
| 1 | Blended Brine (85% $NaCl$ + 15% $MgCl_2$) | 0.7 |
| 2 | $MgCl_2$ | 0.8 |
| 3 | $NaCl$ | 1.8 |

Table 8.6 presents the average percentage of scaling based on mix design formulation. From the results, it can be concluded that Mix 3 had shown no signs of scaling (0%) and was most resistance against scaling. While Mix 1, which has 25% fly ash had shown some scaling (0.1%) compared to Mix 3. The least resistance to scaling was shown by Mix 4, which was considered a poor mix. However, Mix 5, which was also considered a poor mix, has shown only 0.8% average scaling. It performed better than Mix 2, which had an average scaling of 1.1%. It should be noted that only no curing panels of Mix 5 have scaled and panels subjected to other curing methods have shown no signs of scaling. While for Mix 2, panels cured with a chemical compound and the no cured panels, both have shown signs of scaling. The results indicate that the concrete mix which did not incorporate SCMs have shown least scaling when compared to mixes with permissible amount of SCMs.

**Table 8.6 Computer-based photogrammetry results by mix design formulation**

| Ranking | Mix Design Formulation No. | SCM Content (%) | Average Percentage of Scaled Area (%) |
|---|---|---|---|
| 1 | 3 | 0 | 0.0 |
| 2 | 1 | 25 | 0.1 |
| 3 | 5 | 50 | 0.8 |
| 4 | 2 | 50 | 1.1 |
| 5 | 4 | 25 | 3.9 |

## 8.5 Comparison of Phase-I and Phase II Results

The sidewalks panels of Phase-I were placed in November 2019 during cold weather conditions. While the sidewalks from Phase-II were placed in July 2021 during hot weather conditions. The significant difference between these phases is the time between concrete placement and exposure to freezing environment. The panels from Phase-I were exposed to a freezing environment within 5 days of placement. Whereas the panels from Phase-II were not exposed to a freezing environment until around 4 months. From the percentage of scaling determined for all the sidewalk panels from both phases, it can be observed that the panels placed following hot weather concerting procedure perform much better than those placed following cold weather concreting procedures. As the panels placed during summer had more time to mature and gain desired strength, the resistance against freezing environment and deicer exposure. It can be concluded that mature concrete resists the scaling damage caused by freezing temperatures and desires compared to young concrete especially when SCMs are utilized in concrete.

From petrographic study of cores, it was observed that most of the cores from Phase-I panels exhibited a weak top layer which was susceptible to scaling damage. Trowel finishing and inadequate curing are possible reasons for the creation of a weak top layer. However, the cores from Phase-II panels did not exhibit a weak top layer. The cores from Phase-II had localized weak layer due to sub-parallel alignment of elongated or blocky aggregates. Also, most of the cores from Phase-II exhibited mortar flaking.

# 9.0 Recommendations

The recommendations discussed in this chapter are a result of literature review, mix design analysis, placement and finishing practices, various curing methods and deicing methods, scaling resistance tests and petrographic study on hardened concrete. The objective of this chapter is to provide recommendations to prevent or minimize surface deterioration in concrete sidewalks.

To achieve durability in concrete sidewalks, materials, construction, and maintenance practices should be carefully monitored. The performance of concrete sidewalks is highly dependent on the properties of the near surface layer of the concrete body. The placement and construction practices along with the mix design influence the air void system, w/cm ratio and strength of top layer of concrete. An adequate air void system, low w/cm ratio, necessary strength is required in the top surface to resist the exposure to freeze-thaw cycles and deicing agents. Quality assurance (QA) and Quality control (QC) are both important throughout the process of placement and maintenance of the concrete sidewalks to achieve durable sidewalks.

## 9.1 Mix Design Formulation

### 9.1.1 Combined Aggregate System

1. **The combined aggregate system of the fine and coarse aggregates must satisfy the tarantula curve and Shilstone workability coarseness chart mentioned in Section 3.1**
   The tarantula curve and workability coarseness chart must be satisfied as the gradation of aggregates influences the workability of concrete mix. However, the petrographic study results indicate that the total aggregate content was lower than the mix design value for all the mix formulations examined.

### 9.1.2 Paste System

1. **According to AASHTO PP 84-17, the paste content must be lower than or equal to 28% to decrease the cracking tendency of concrete**
   By keeping the paste content lower than AASHTO PP 84 recommendation of 28%, the workability of concrete is achieved and tendency to crack due to drying shrinkage also decreases.
2. **According to the interim revision to the 2020 edition of the Commonwealth of Massachusetts Department of Transportation Standard Specifications (Subsection 701), paste content to aggregate void content ratio (PC/VC) should be between 1.25 and 1.75**
   The volume of paste should adequately fill the voids and provide sufficient separation between aggregate particles, thereby promoting workability and bonding of particles.

### 9.1.3 Air Void System

1. **According to ACI 201.2R, the recommended air content for a concrete mix with a ¾ in. nominal maximum aggregate size and Exposure Class F3 (Concrete exposed to freezing and thawing conditions as well as deicing chemicals) is 7 ± 1.5%**
   Incorporation of an adequate air void system in concrete will result in higher resistance against scaling. The hardened air content determined from petrographic analysis is withing the limit for Mix 1,2 and 3 which were intentionally air entrained.
2. **According to ACI 201.2R, the accepted maximum spacing factor in concrete is 0.008 in. and minimum specific surface area is 600 $in^2/in^3$ for resistance to freezing and thawing**
   The air void system parameters determined through petrographic study have indicated that the mixes which were intentionally air entrained have the spacing factor below 0.008 and specific surface area greater than 600 $in^2/in^3$. While the mixes which were not air entrained exceeded the recommended values.

### 9.1.4 Supplementary Cementitious Materials

1. **According to ACI 201.2R, the maximum percent of total cementitious materials by mass for Exposure Class F3 (Concrete exposed to freezing and thawing conditions as well as deicing chemicals) for Fly ash is 25% and Slag is 50%**
   The fly ash and slag incorporated in the concrete must comply with ASTM C618 and ASTM C989 respectively. All the mix designs incorporating SCMs had total SCM content aligning with recommended values expect for Mix 5 which had 50.2% slag. Mix 1 and 5 with 25% fly ash and 50.2% slag respectively had shown better performance. However, Mix 2 and 4 which contained 49.8% slag and 25% fly ash respectively have shown least resistance to scaling especially Mix 4 which had highest percentage of scaled area. It should be noted that Mix 3 which contained no SCM had shown no signs of scaling irrespective of curing and deicing methods.

### 9.1.5 Water-Cementitious Ratio

1. **According to ACI 201.2R, the maximum w/cm recommended is 0.45 for Exposure Class F3 (Concrete exposed to freezing and thawing conditions as well as deicing chemicals)**
   The w/cm ratio determined for all the mix design formulations were below 0.45 except of Mix 4 which had a w/cm ration of 0.51. However, the w/cm ratios determined for each mix from petrographic analysis was less than 0.45. This could be a result of maturity of concrete and hot weather conditions. A higher w/cm ratio is not recommended as it decreases the strength of the concrete. A lower w/cm ratio results in low permeability and greater durability. With the recommended w/cm ratio, the concrete can achieve an adequate compressive strength and lower permeability which ensures a durable concrete.

### 9.1.6 Compressive Strength

1. **According to ACI 201.2R, the minimum compressive strength before exposure to repeated cycles of freezing and thawing in presence of deicing salts is 4500 psi**
   When the concrete achieves recommended strength before freezing will reduce the occupiable volume by freezable water in saturated concrete due to formation of hydration products. Also, concrete with sufficient tensile strength of paste will resist deterioration due to freezing and thawing better. The 91 days compressive strength determined for all mixes was greater than 4500 psi which is sufficient as the sidewalks in the field were not exposed to freezing environment until around 120 days after placement.

### 9.1.7 Chemical Admixtures

1. **Water-reducing and air entraining admixtures are recommended for a workable and durable concrete**
   Using an air entraining admixture (AASHTO M 154 – P-AEA) ensure a good air void system which satisfies the recommended air void parameters. The mixtures which were not air entrained tend to have large irregularly shaped entrapped air voids which do not satisfy the recommendations for a durable concrete as observed in Mix 4. The use of a water-reducing admixture (AASHTO M 194 – Type A) will compensate for the increased water demand during hot weather conditions and a lower w/cm ratio can be achieved.

## 9.2 Placement, Finishing and Curing Practices

### 9.2.1 Concrete Batching and Delivery

1. **Additional water added in transit or on site should be according to AASHTO M 157**
   The water added in transit or on site should not exceed the maximum allowable w/cm ratio and slump as per the mix design. As ambient temperatures are high during the hot weather, the water demand increase. However, addition of extra water should be monitored because as w/cm increases, the strength of concrete decreases and permeability increases which directly affects the durability of concrete. During hot weather concreting, chilled water can be used to reduce the temperature of concrete during placement in accordance with ACI 305-20.

### 9.2.2 Concrete Placement and Finishing

The concrete placement and finishing must be done following Sections 5.3.2 and 5.3.4 of ACI 301-16 and also National Ready Mix Concrete Association (NRMCA) Flatwork Best Practices. In addition to the specifications, the recommendations mentioned in ACI 305R-20 should also be followed to ensure a durable concrete is achieved from hot weather concreting procedure. When the high ambient temperatures, high wind speeds, or low relative humidity are anticipated during placement, then precautions must be taken as these factors increase the rate of evaporation.

1. **Considerations must be made to reduce the risk of early-age thermal cracking and lower strength**
   Thermal cracking can occur when temperature differentials in concrete induced thermal stresses leading to cracking. High concrete temperatures decrease the later age strength of concrete. Therefore, precautions must be taken during hot weather conditions to maintain the temperature of the concrete. The strength reductions due to elevated temperatures can be compensated to some extent by use of SCMs as the hydration process is slower and lower heat of hydration is produced. The temperature of plastic concrete can be decreased by using chilled mixing water or adding ice to mixing water, or cooling aggregates by sprinkling water (but not increase the w/cm ratio).
2. **Finishing practices must be following the recommendations mentioned in Section 2.3 and in accordance with ACI 201.2R**
   Over-finishing of the concrete surface must be avoided. A weak layer with high w/cm in the top surface will be created if the bleed water is worked back into the surface. The timing of finishing must be carefully identified as it affects the durability of concrete especially when SCMs are used. The setting time is delayed when SCMs are used.
3. **Sidewalks should be sufficiently mature prior to exposure to freezing, thawing, and de-icing.**
   If placed in hot or cold conditions, ACI practices (ACI 305R-20 Guide to Hot Weather Concreting and ACI 306R-16 Guide to Cold Weather Concreting) must be met *(14)*.

### 9.2.3 Concrete Curing

The concrete curing must be done following Sections 5.3.6 of ACI 301-16. ACI 305R-20 provides recommendations for curing and protection of concrete placed under hot weather conditions. Hot weather conditions are typically defined as air temperatures exceeding 80 degrees F and low humidity. Under these conditions, measures should be taken to prevent excessive evaporation that may result in drying, shrinkage, and cracking of concrete surfaces. Concrete surfaces should not be allowed to become surface dry while transitioning from plastic to hardened states. Curing procedures may involve the use of approved sealants and curing membranes and moist curing. If sealants and curing membranes are used, reapplication may be required per manufacturer recommendations. If moist curing is conducted, moisture must be maintained for at least seven days (through either periodic wetting or using curing blankets). Care should be taken to prevent staining of the concrete surface when using blankets. The curing methods should be done by strictly following the guidelines form ACI 301-16 and ACI 305R-20 to achieve an adequately durable concrete which resists deterioration due to freezing environment and deicing chemicals. Cold weather concreting practices are detailed in the Phase 1 companion report to this effort *(1)*.

1. **Considerations must be made to reduce the risk of surface drying**
   During hot weather conditions, there is an increased risk of plastic shrinkage cracking. When the rate of evaporation is faster than rate at which surface moisture is replenished through bleeding or initial curing, surface dying. The rate of

evaporation is typically faster during hot weather conditions. Use of SCMs will reduce rate of bleeding, therefore increasing the risk of plastic shrinkage cracking. Adequate curing is required to replenish the lost surface moisture and aid in the hydration process.

2. **Moisture curing with a saturated cover must be done according to ACI 305R-16**
In accordance with ACI 308R, moisture curing with saturated covers must be done for at least 7 days after placement to ensure a proper strength gain through hydration process. When moist curing is done right, it minimizes early-age drying shrinkage and increases strength and durability of concrete. The saturated cover should not be allowed to dry during curing, as the burlap tends to absorb moisture from concrete when it's dry. Extended curing must be done when concrete contains SCMs as the rate of strength gain is slow. Moisture curing is beneficial during hot weather conditions because the surface moisture lost due to evaporation is replenished by the wet burlap.
3. **Curing compounds must conform with ASTM C1315 and the instructions given by the manufacturer must be followed**
The time application of curing compounds is an important factor. The curing compounds must be applied strictly following manufacturer's instructions. Due to early application of curing compounds, the bleed water is trapped under the membrane leading to a weak top layer, especially with concrete containing SCMs as the rate of bleeding is decreased. During hot weather conditions, the curing compounds may not be effective if they are applied after the surface moisture is lost due to evaporation because curing compounds do not provide additional moisture but retain the moisture in concrete for hydration.

## 9.3 Winter Treatment and Maintenance

The sidewalks in this study were placed around 120 days prior to beginning freezing conditions. Most of the sidewalks did not show any signs of scaling after being subjected to 18 freeze-thaw cycles and deicing agents during the first winter after placement. The following recommendations are made from the observations made during the winter treatment and maintenance.

1. **The time between placement of sidewalks and first freeze influences the salt scaling resistance of concrete**
Due to the placement of sidewalks during summer, the concrete (most of the sidewalks) matured and gained enough strength to resist the effect of freeze-thaw cycles and application of deicing agents. It is recommended that the concrete gains a minimum of 4500 psi strength before freezing, thawing, or de-icing cycles begin. The sidewalks in this study were placed 120 days prior to commencement of freezing, thawing, or de-icing cycles, following the proper curing and placement practices and allowing the concrete to mature before application of deicing agents. These procedures resulted in favorable scaling resistance.
2. **The use of deicing agents with magnesium chloride ($MgCl_2$) showed better results compared to NaCl**

The two deciding agents used in the study, 30% by wt. $MgCl_2$ and Blended Brine with 15% $MgCl_2$, were less aggressive on the sidewalks compared to NaCl. The sidewalks subjected to NaCl have shown more scaling compared to the other two deicing agents. Furthermore, sidewalks treated using NaCl showed a higher chloride content than those treated using MgCl2. The presence of chlorides is detrimental to concrete durability performance.

# 10.0 References


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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/dehc-xy98**