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COMPARISON OF CONVENTIONAL AND INTERNALLY CURED CONCRETE BRIDGE DECKS IN UTAH: MOUNTAIN VIEW CORRIDOR PROJECT

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16. Abstract
The objectives of this research were to 1) monitor in-situ moisture and diffusivity for both conventional concrete and concrete containing pre-wetted lightweight fine aggregate (LWFA), 2) compare deck performance in terms of early-age cracking, compressive strength, and chloride ingress, and 3) compare concrete properties in terms of compressive strength, chloride permeability, elastic modulus, and water content in the laboratory using cylinders cast in the field at the time of deck construction. The research involved field and laboratory evaluations of four newly constructed bridge decks located in northern Utah, two constructed using conventional concrete and two constructed using pre-wetted LWFA to promote internal curing. Data from sensors embedded in the concrete decks indicate that the volumetric moisture content and electrical conductivity of the internally cured concrete were higher than those of the conventional concrete during the 2-year monitoring period. Laboratory data indicate that the internally cured concrete was weaker at 1 year following deck construction, passed less current in rapid chloride permeability testing, had a lower modulus in free-free resonant testing, and had a higher gravimetric moisture content than the conventional concrete. In the field, the internally cured concrete was weaker at 2 years in Schmidt rebound hammer testing and exhibited greater chloride concentrations at both 1 and 2 years than the conventional concrete. The conventional concrete bridge decks had more reflection cracking from the joints between the underlying pre-cast half-deck panels than the internally cured concrete decks.

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LIST OF ACRONYMS

ASTM	American Society for Testing and Materials
BYU	Brigham Young University
CH	calcium hydroxide
C-S-H	calcium-silicate-hydrate
DOT	department of transportation
LWFA	lightweight fine aggregate
NB	northbound
NE	northeast
NW	northwest
RCPT	rapid chloride permeability test
SB	southbound
SE	southeast
SW	southwest
SAP	super-absorbent polymer

EXECUTIVE SUMMARY

The objectives of this research were to 1) monitor in-situ moisture and diffusivity for both conventional concrete and concrete containing pre-wetted lightweight fine aggregate (LWFA), 2) compare deck performance in terms of early-age cracking, compressive strength, and chloride ingress, and 3) compare concrete properties in terms of compressive strength, chloride permeability, elastic modulus, and water content in the laboratory using cylinders cast in the field at the time of deck construction. The research involved field and laboratory evaluations of four newly constructed bridge decks located in northern Utah, two constructed using conventional concrete and two constructed using pre-wetted LWFA to promote internal curing.

Data from sensors embedded in the concrete decks indicate that the average volumetric moisture content of the internally cured concrete was 2 to 4 percentage points higher than that of the conventional concrete for the first 1 year following deck construction but less than 2 percentage points at 2 years. Although the internally cured concrete decks had a consistently higher average moisture content, the average electrical conductivity values of the internally cured concrete decks were not consistently higher than those measured on the conventional concrete decks until after 6 months when the average electrical conductivity of the internally cured decks was 38 to 50 percent greater than that of the conventional concrete decks.

Laboratory compressive strength data indicate that, for the first 6 months following deck construction, the two concrete mixtures exhibited very similar strength gain characteristics. However, at 1 year, the conventional concrete was stronger than the internally cured concrete by an average of 12.9 percent. In rapid chloride permeability testing, the internally cured concrete consistently passed between 13.1 and 17.5 percent less current than that passed by the conventional concrete. Laboratory free-free resonant testing at 1 year showed that the modulus of the internally cured concrete was 3.9 percent lower, on average, than that of the conventional concrete. For the tested specimens, the gravimetric moisture content of the internally cured concrete was 0.5 percentage points higher, on average, than that of the conventional concrete.

In the field, Schmidt rebound hammer testing showed similar strengths for both deck types at 1 year but showed that the internally cured concrete was weaker than the conventional concrete at 2 years. On average, the internally cured concrete exhibited between 1.4 and 15.7

percent and between 12.3 and 46.2 percent greater chloride concentration, depending on the depth interval, than the conventional concrete at 1 and 2 years, respectively. On average, at 5 months, 8 months, 1 year, and 2 years, the conventional concrete bridge decks had 4.8, 6.6, 2.5 and 1.3 times more cracking, respectively, than the internally cured concrete decks. During the 1-year and 2-year distress surveys, very distinctive reflection cracks from the joints between the underlying pre-cast half-deck panels were observed on all of the decks.

1.0 INTRODUCTION

1.1 Problem Statement

The long-term performance of concrete bridge decks is a function of the quality of concrete curing, especially in cold regions. During the weeks immediately following deck construction, high degrees of moisture saturation are desirable to ensure good concrete curing necessary for the concrete to develop both strength and durability, including a reduction in cracking susceptibility (*1*). One method of maintaining high degrees of moisture saturation in concrete immediately following deck construction is the use of pre-wetted, lightweight fine aggregate (LWFA). LWFA has a much higher absorption than conventional aggregate, and the absorbed water is gradually released into the cement paste over time to extend the cement hydration process even after the bridge deck is opened to traffic (*2*). Because both autogenous and drying shrinkage are reduced, shrinkage cracking is also minimized. For these reasons, the process of "internal curing" is expected to yield a more durable concrete bridge deck.

Past laboratory research has investigated the degree of cement hydration, compressive strength, chloride permeability, shrinkage cracking, and service life of concrete containing pre-wetted LWFA (*2, 3, 4, 5, 6, 7*). However, field studies on concrete bridge decks with pre-wetted LWFA are still few in number. The departments of transportation (DOTs) from the states of Indiana, New York, and Ohio have each implemented the use of LWFA in bridge deck construction. While more bridge decks incorporating LWFA are currently under construction or being planned, as of 2011, the Indiana DOT had constructed two bridge decks, the New York DOT had constructed nine bridge decks, and the Ohio DOT had constructed one bridge deck (*7, 8, 9*). Although site inspections have been conducted on bridge decks with LWFA, information regarding in-situ deck properties such as moisture and diffusivity over time is not available in the literature.

1.2 Research Objectives and Scope

The objectives of this research were to 1) monitor in-situ moisture and diffusivity for both conventional concrete and concrete containing pre-wetted LWFA, 2) compare deck performance in terms of early-age cracking, compressive strength, and chloride ingress, and 3) compare concrete properties in terms of compressive strength, chloride permeability, elastic modulus, and water content in the laboratory. The research involved field and laboratory evaluations of four newly constructed bridge decks located in the Mountain View Corridor in West Jordan, Utah. Two were constructed using conventional concrete, and two were constructed using pre-wetted LWFA to promote internal curing. Sensors for measuring the in-situ moisture content, temperature, and electrical conductivity were embedded in each concrete bridge deck. Deck performance was compared in terms of surface cracking at 5 months, 8 months, 1 year, and 2 years following deck construction. Concrete cylinders cast in the field at the time of construction were prepared to evaluate compressive strength, chloride permeability, elastic modulus, and water content. Field monitoring and laboratory testing were performed to determine the benefits of using LWFA to promote internal curing of concrete bridge decks in Utah.

1.3 Report Outline

This report contains five chapters. Chapter 1 introduces the research, and Chapter 2 provides background information about the use of LWFA in concrete. Chapters 3 and 4 present the research procedures and results, respectively, while Chapter 5 gives conclusions and recommendations based on the research findings.

2.0 BACKGROUND

2.1 Overview

The following sections describe the effects of LWFA on the curing and cracking of concrete and on the performance of concrete bridge decks.

2.2 Concrete Curing

Proper curing of concrete is required for optimum performance in any environmental condition or application (*10*). Curing involves maintaining sufficient internal moisture within concrete necessary for it to develop appropriate levels of strength and durability. During the curing process, water reacts with the cementitious materials in concrete to form two main hydration products, calcium-silicate-hydrate (C-S-H), the primary source of strength, and calcium hydroxide (CH) (*11*). A sufficient amount of water well distributed throughout the concrete matrix is necessary to ensure a high degree of cement hydration.

Internal curing, accomplished through the use of pre-wetted LWFA, provides small reservoirs of additional water located in the permeable pores of the LWFA. This water, which is in addition to the free water necessary to achieve the specified water-cementitious materials ratio for the given concrete mixture, allows the concrete to hydrate longer and therefore to become more durable and less permeable (*11, 12*). Pre-wetting of the LWFA in the concrete batching process is necessary for the LWFA to absorb the required water before being mixed with the other concrete ingredients. Lightweight aggregates exhibit absorption values typically between 10 and 20 percent, depending on the aggregate type. For example, absorption percentages following 24 hours of saturation for expanded shale, clay, and slate range from 15 to 30 percent, 10 to 20 percent, and 6 to 12 percent, respectively (*7*). The use of expanded aggregates can also improve aggregate-paste bonding in concrete due to the rough surface texture and pozzolanic mineralogy (*13*).

2.3 Concrete Cracking

Through the reaction of cement and water and subsequent formation of C-S-H and CH, concrete undergoes a reduction in volume (*10*). As the concrete begins to set, the cement paste develops resistance to further deformation and volume reduction. In the absence of additional curing water, the concrete self-desiccates, reducing the degree of saturation of the capillary voids within the concrete microstructure (*14*). Particularly in concrete mixtures with low water-cementitious materials ratios, water available from external curing methods such as ponding, fogging, misting, and wet burlap applications cannot sufficiently penetrate the concrete, causing the interior of the concrete to self-desiccate even when the exterior is kept moist (*15*).

As concrete self-desiccates during the hydration process and/or experiences water loss due to evaporation, the concrete develops shrinkage stresses, which, when restrained, lead to the introduction of tensile stresses within the concrete. When the induced tensile stresses exceed the tensile strength of the concrete, cracking occurs (*16*). The addition of pre-wetted LWFA into the concrete mixture, which ensures a source of readily available internal water, reduces the development of tensile stresses by facilitating the movement of water from the aggregate into the cement paste during the hydration process. Because of the larger pore sizes in the LWFA, as compared to the pore sizes within the hydrating cement, water is drawn out of the LWFA to the surrounding cement paste, which effectively prevents internal drying of the concrete and thereby reduces shrinkage stresses and the occurrence of early-age cracking (*12*, *13*).

2.4 Concrete Bridge Deck Performance

Cracking of concrete bridge decks increases the potential for deck deterioration, especially in cold climates where chloride-based deicing salts are regularly applied to the deck surface as part of winter maintenance activities. Cracking provides a direct pathway for chloride ions to penetrate the deck and accumulate in the vicinity of the embedded reinforcing steel. When critical chloride concentrations are exceeded, corrosion of the reinforcing steel can commence, leading to structural deterioration of the bridge deck (*17*). Because corrosion of reinforcing steel can be an expensive problem to mitigate (*18*), providing low-permeability concrete and preventing cracking of bridge decks are critical design objectives (*19*). To this end, the use of pre-wetted LWFA has been shown to densify the microstructure of concrete, reduce permeability, and reduce shrinkage cracking, thereby extending deck service life (*1, 20*).

Although the use of pre-wetted LWFA on bridge decks is a relatively new approach, researchers have quantified potential extensions in service life through numerical modeling. In a case study to determine the benefits of internal curing, the use of LWFA was projected to extend the life of high-performance concrete bridge decks by more than 20 years. The research proposed that a conventional concrete deck would have a service life of 22 years, a high-performance concrete deck without pre-wetted LWFA would have a service life of 40 years, and a high-performance concrete deck with pre-wetted LWFA to promote internal curing would have a service life of 63 years (*21*). By increasing the service life of a deck, local agencies and state DOTs can significantly lower the overall life-cycle cost of a bridge through reductions in maintenance requirements and rehabilitation efforts.

2.5 Summary

Proper curing of concrete is required for optimum performance in any environmental condition or application. Internal curing, accomplished through the use of pre-wetted LWFA, provides small reservoirs of additional water located in the permeable pores of the LWFA. This water, which is in addition to the free water necessary to achieve the specified water-cementitious materials ratio for the given concrete mixture, allows the concrete to hydrate longer and therefore to become more durable and less permeable.

In the absence of additional curing water, the concrete develops shrinkage stresses, which, when restrained, lead to the introduction of tensile stresses within the concrete. When the induced tensile stresses exceed the tensile strength of the concrete, cracking occurs. The addition of pre-wetted LWFA into the concrete mixture, which ensures a source of readily available internal water, reduces the development of tensile stresses by facilitating the movement of water from the aggregate into the cement paste during the hydration process, effectively preventing internal drying of the concrete and thereby reducing shrinkage stresses and the occurrence of early-age cracking.

Cracking of concrete bridge decks increases the potential for deck deterioration, especially in cold climates where chloride-based deicing salts are regularly applied to the deck surface as part of winter maintenance activities. Because corrosion of reinforcing steel can be an expensive problem to mitigate, providing low-permeability concrete and preventing cracking of bridge decks are critical design objectives. To this end, the use of pre-wetted LWFA has been shown to densify the microstructure of concrete, reduce permeability, and reduce shrinkage cracking, thereby extending deck service life and significantly lowering the overall life-cycle cost of a bridge.

3.0 PROCEDURES

3.1 Overview

The following sections provide a site description and discuss bridge design, deck instrumentation, deck construction, laboratory testing, and field testing relevant to this project.

3.2 Site Description

In the spring of 2012, the Mountain View Corridor project team of the Utah DOT constructed four new bridges in West Jordan, Utah, including two at Dannon Way and two at 8200 South. At each location, one deck served northbound (NB) traffic and one deck served southbound (SB) traffic. One bridge deck at each location was constructed using a conventional concrete mixture, and one was constructed using a concrete mixture containing a portion of pre-wetted LWFA to facilitate internal curing. Figure 3-1 shows an aerial image of the location of each bridge and indicates the type of concrete utilized at each site.

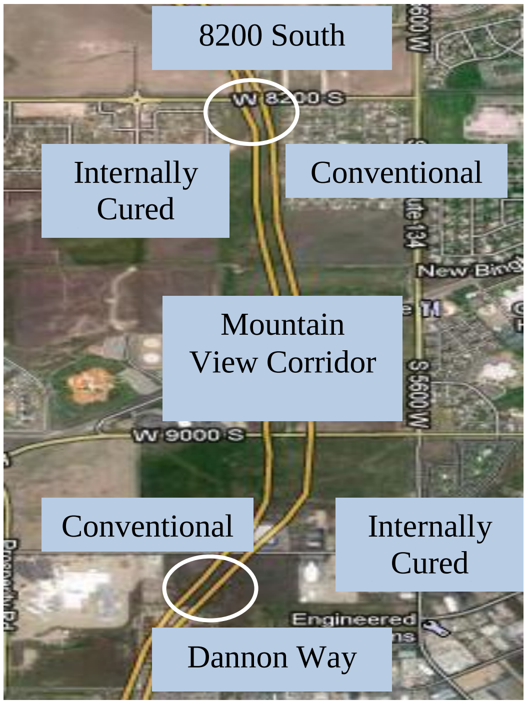

Figure 3-1 Bridge locations and concrete types.

3.3 Bridge Design

Each of the four bridges shared a similar design. Each structure incorporated five pre-stressed, pre-cast, single-span concrete girders, as shown in Figure 3-2. Pre-cast half-deck concrete panels, with a width of 7 ft 11 in. and a length of 8 ft, were placed between the girders along the majority of the length of the bridge; near each abutment, shorter panels were used as necessary. Each panel extended 6 in. onto each girder, as shown in Figure 3-3, and was supported by rigid foam that was placed between the girder and the pre-cast panel as illustrated in Figure 3-4; by design, the panels were not connected across the transverse butt joints between them on any of the decks but were simply placed close together as shown in Figure 3-5 for the Dannon Way NB bridge. A mat of No. 5 reinforcing steel was then placed over the tops of the half-deck panels and girders in preparation for concrete placement across the deck. The total design width of each bridge deck was 50 ft 10 in., sufficient for two lanes of traffic, and the design deck lengths ranged from 119 ft 7-3/8 in. to 128 ft 1-5/8 in. A minimum concrete cover depth of 2.5 in. over the top mat of reinforcing steel was specified for the project.

Figure 3-2 Pre-stressed, pre-cast, single-span concrete girders.

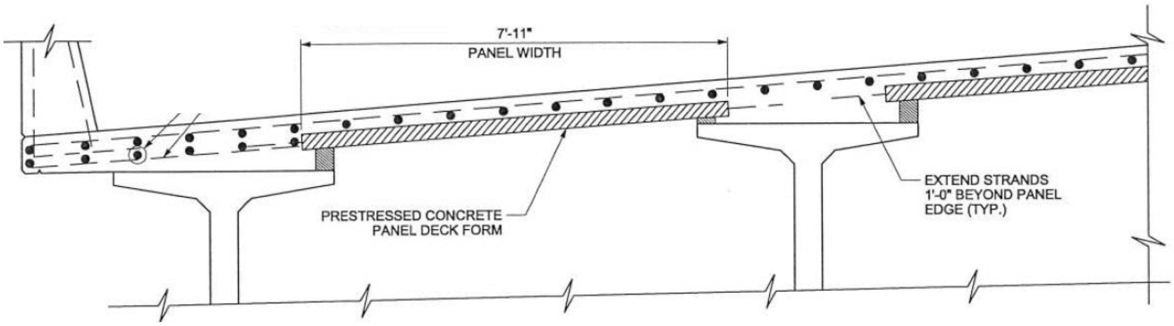

Figure 3-3 End view of pre-cast half-deck concrete panels.

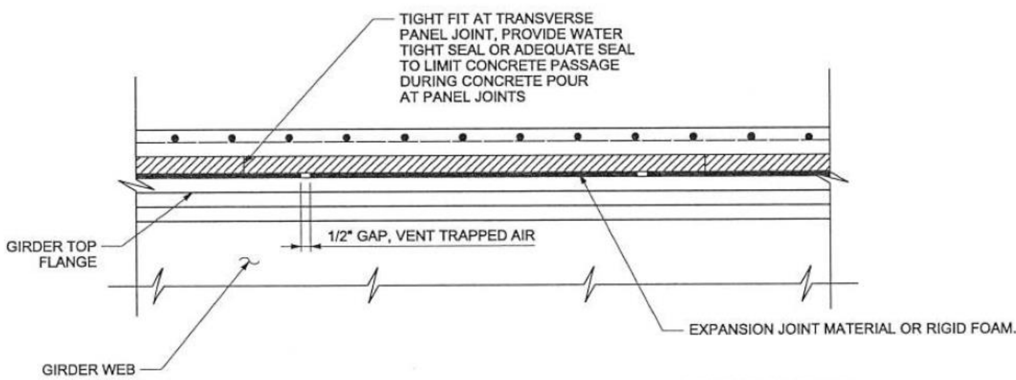

Figure 3-4 Side view of pre-cast half-deck concrete panels.

Figure 3-5 Half-deck concrete panels.

3.4 Deck Instrumentation

Prior to concrete placement, each of the four bridge decks was instrumented with three sensors connected to a data logger equipped with cellular telephone service. The sensors were checked in a uniform sand mixture within the laboratory to ensure consistent readings from each sensor prior to installation. Sensors were placed approximately 15 ft apart, with two sensors in the wheel path of the right lane and one sensor in the wheel path of the left lane on each bridge. The actual sensor layouts for each deck are provided in Appendix A. Placing the sensors in this pattern ensured that they would each be embedded in concrete from different ready-mix trucks and allowed the data to be more representative of the full deck. The sensors measured the volumetric moisture content, temperature, and electrical conductivity of the concrete on hourly intervals. The moisture content reflected the amount of water in the concrete, and temperature measurements allowed for documentation of environmental conditions. Electrical conductivity was a useful surrogate measure of diffusivity, since electrical conductivity is affected by many of the same factors that affect the diffusivity of porous media.

The three sensors were installed in each deck at the level of the top mat of reinforcing steel, as displayed in Figure 3-6. The sensor cables were routed out of each deck through the nearest diaphragm wall to a location generally between the concrete girders, as illustrated in

Figure 3-7, where they could be conveniently terminated in a secure box. A battery-powered data logger was mounted inside the box, as shown in Figure 3-8, to facilitate data collection.

Figure 3-6 Sensor installed at level of top mat of reinforcement.

Figure 3-7 Mounting of data logger box on diaphragm wall.

Figure 3-8 Data logger mounted in box.

Individual sensors were then protected with temporary wooden covers through the duration of the construction process. After the concrete was placed and consolidated around a given sensor, the wooden cover was removed for deck finishing.

In addition, air temperature, relative humidity, and precipitation gauges were mounted on the bridges to monitor the ambient conditions to which the bridge decks were exposed. Air temperature and relative humidity gauges were mounted at the 8200 South SB and Dannon Way NB bridges, while precipitation gauges were mounted at the 8200 South NB and Dannon Way SB bridges. A typical installation is shown in Figure 3-9.

Figure 3-9 Precipitation gauge mounted on bridge.

3.5 Deck Construction

Table 3-1 shows the concrete mixture designs approved by the Utah DOT for use in construction of the conventional and internally cured concrete bridge decks evaluated in this research; the 28-day concrete compressive strength specified by the Utah DOT for both mixtures was 4,000 psi. The mixture designs were identical to each other except for the inclusion of the pre-wetted LWFA in the internally cured concrete. The internally cured concrete was designed to provide 7 lb of absorbed water per 100 lb of cementitious material (*22*), which resulted in a 30 percent replacement of the fine aggregate with pre-wetted LWFA by volume.

Table 3-2 provides additional concrete mixture design parameters. The unit weight is the theoretical value computed from the mixture design in each case. Although the inclusion of the LWFA lowers the theoretical unit weight by 5 pcf, the internally cured concrete mixture is still considered a normal-weight concrete mixture. The additional water introduced to the internally

Table 3-1 Concrete Mixture Designs

Ingredient	Conventional (yd^3)	Internally Cured (yd^3)
Type II/V Cement	0.09	0.09
Class F Fly Ash	0.03	0.03
Water	0.16	0.16
Coarse Aggregate (3/4 in. Nominal)	0.39	0.39
Fine Aggregate	0.27	0.16
Lightweight Fine Aggregate	0.00	0.11
Air Entrainment	0.06	0.06
Total	1.00	1.00

Table 3-2 Concrete Mixture Design Parameters

Description	Conventional	Internally Cured
Unit Weight (pcf)	137.3	132.3
Water-Cementitious Materials Ratio	0.44	0.44
Target Slump (in.)	3.5	3.5
Low-Range Water Reducer (oz/cwt)	3.0	3.0
Fine Aggregate Absorption (%)	1.20	1.20
LWFA Absorption (%)	-	15.0

cured concrete through the use of pre-wetted LWFA does not increase the water-cementitious materials ratio of the mixture because the water is initially absorbed by the LWFA.

For the internally cured concrete, the LWFA was pre-wetted for a minimum of 2 days at the batch plant using a sprinkling system to achieve a minimum moisture content of 15 percent, and excess water was then allowed to drain prior to concrete batching, as shown in Figure 3-10.

Figure 3-10 Pre-wetting of lightweight fine aggregate at concrete batch plant.

This process ensured that the LWFA was near saturation when mixed with the other concrete ingredients.

The concrete bridge deck construction procedures were consistent for each bridge. The concrete placed on all four decks was supplied by the same concrete producer and placed and finished by the same construction crew. A target slump of 3.5 in. and 6 percent air content were specified for each concrete bridge deck; according to Utah DOT records, the average slump measured during construction ranged from 3.25 to 4.0 in., and air content ranged from 5.7 to 6.4 percent. Detailed slump and air content data are provided in Appendix B. Concrete was pumped into place, consolidated using internal vibrators, and uniformly spread using a Bidwell paver.

To allow the girders to reach their fully deflected positions before the diaphragm walls were cast around the ends of girders, the deck was placed first. To accomplish this phasing, metal shoring was placed in the transverse direction near one end of the deck, as depicted in Figure 3-11, to establish an initial starting point for the deck pour. Following concrete placement from that point to the other end of the bridge span and into the opposite diaphragm wall, the contractor returned to the original starting point to place the remaining deck section and diaphragm wall. In this process, the metal shoring was often only partially removed or allowed to remain entirely in place for convenience. A curing agent was sprayed onto the deck following concrete placement, and the deck was then covered with plastic, as shown in Figure 3-12, for a specified 14-day curing period, after which the deck was fully exposed to ambient conditions. After construction, concrete cover depths were measured at each sensor location. As

Figure 3-11 Metal shoring used in deck construction process.

Figure 3-12 Covered bridge deck during curing.

documented in Appendix B, average cover depths were 3.4 and 2.8 in. on the Dannon Way SB and Dannon Way NB decks, respectively, and 2.7 and 2.4 in. on the 8200 South NB and 8200 South SB decks, respectively.

3.6 Laboratory Testing

As described in the following sections, laboratory testing was performed to determine the compressive strength, chloride permeability, elastic modulus, and moisture content for the two concrete mixtures. These tests were performed on concrete cylinders cast during placement of the four bridge decks. Specifically, 10 concrete cylinders, each 4 in. in diameter and 8 in. in height, were cast in accordance with American Society for Testing and Materials (ASTM) C31 (Standard Practice for Making and Curing Concrete Test Specimens in the Field) using concrete sampled from each of the three sensor locations, for a total of 30 concrete cylinders from each deck. The three sensor locations were selected as the sampling sites in order to test the same concrete in which the sensors were embedded. The cylinders were cured in a fog room for 28 days, after which eight of the 10 cylinders were relocated onto the laboratory bench in open air to more closely simulate field curing conditions. The average temperature and relative humidity measured in the laboratory were 73°F and 21 percent, respectively. The remaining two cylinders were stored in the fog room for 1 year as control samples to simulate an extended curing period for the concrete.

3.6.1 Compressive Strength Testing

As an indication of the overall quality of the concrete, compressive strength testing was performed in accordance with ASTM C39 (Standard Test Methods for Compressive Strength of Cylindrical Concrete Specimens) on one cylinder from each sensor location on each bridge deck at 7 days, 28 days, 58 days, 3 months, 6 months, and 1 year. The cylinders tested at 7 and 28 days were cured in the fog room, the cylinders tested at 58 days were cured for 28 days in the fog room and then subjected to 30 days of freeze-thaw cycling, the cylinders tested at 3 and 6 months were cured for 28 days in the fog room and then placed on the laboratory bench in open air, and the cylinders tested at 1 year included a set that was cured for 28 days in the fog room and then placed on the laboratory bench in open air and a set that was cured in the fog room for the entire

1 year. All cylinders removed directly from the fog room for compressive strength testing were removed several hours prior to testing to allow the surfaces of the cylinders to dry.

As illustrated in Figure 3-13, the cylinders were capped with sulfur and tested at a strain rate of 0.05 in./minute at the Brigham Young University (BYU) Highway Materials Laboratory. For the freeze-thaw cycling, each cycle consisted of 12 hours of freezing at 4°F and 12 hours of thawing in the fog room at 80 to 85°F, based generally on guidelines from ASTM C666 (Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing). After the final freeze-thaw cycle, the cylinders were capped and tested in compression, again following ASTM C39.

Figure 3-13 Compressive strength testing.

3.6.2 Rapid Chloride Permeability Testing

To evaluate the ability of the concrete to resist chloride penetration, the rapid chloride permeability test (RCPT) was performed on one cylinder from each sensor location on each bridge deck at 28 days, 6 months, and 1 year, following ASTM C1202 (Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration). The cylinders tested at 28 days were cured in the fog room, the cylinders tested at 6 months were cured for 28 days in the fog room and then placed on the laboratory bench in open air, and the cylinders tested at 1 year included a set that was cured for 28 days in the fog room and then placed on the laboratory bench in open air and a set that was cured in the fog room for the entire 1 year.

RCPT samples were cut from the middle of each cylinder to a thickness of 2 in. using a masonry saw so that up to three samples per cylinder could be obtained. The samples were then saturated for a period of 24 hours in de-aired, de-ionized water. After the conditioning period was complete, the opposite faces of each specimen were exposed to solutions of sodium hydroxide and sodium chloride while a 60-V potential was imposed over the length of the specimen. During the 6-hour test, the total charge, in coulombs, that passed through the specimen was measured, and the concrete permeability was classified according to the threshold values presented in Table 3-3. The RCPT apparatus is shown in Figure 3-14.

Table 3-3 Rapid Chloride Permeability Classifications

Concrete Permeability	Charge Passed (coulombs)
Very Low	< 1000
Low	1000 to 2000
Moderate	2000 to 4000
High	> 4000

Figure 3-14 Rapid chloride permeability testing.

3.6.3 Modulus Testing

To assess the stiffness of the conventional and internally cured concrete mixtures, non-destructive free-free resonant tests were conducted on the same cylinders that were subjected to compressive strength testing at 1 year. The free-free resonant testing was performed immediately before the strength testing and included a set of cylinders that was cured for 28 days in the fog room and then placed on the laboratory bench in open air and a set of cylinders that was cured in the fog room for the entire 1 year for each of the four bridge decks.

For testing, each concrete cylinder was placed on a stand as depicted in Figure 3-15, and the upper end of the cylinder was tapped lightly with a small hammer instrumented with a load cell that triggered data acquisition by an attached computer. An accelerometer mounted in a foam disk placed beneath the cylinder measured the amplitude and frequency of the stress waves induced in the concrete by the hammer strike, and the data were recorded and analyzed by the computer. Three tests were performed on each end of each concrete cylinder, for a total of six measurements per cylinder. The length, diameter, and weight of each cylinder were also recorded, with the length and diameter being measured at three separate locations each. These

Figure 3-15 Free-free resonant testing.

values together with the average frequency measurement were used in the calculation of elastic modulus according to Equation 3-1:

$$E = \frac{\frac{\gamma}{32.2} \cdot (2 \cdot l \cdot f)}{144} \qquad (3\text{-}1)$$

where:

E = elastic modulus (psi)
γ = density (lb/ft^3)
l = length (in.)
f = frequency (Hz)

3.6.4 Moisture Content Testing

To enable a comparison of the moisture contents within the conventional and internally cured concrete mixtures, the cylinders subjected to compressive strength testing at 1 year were retained for further investigation. The sulfur capping material was removed from each of the tested cylinders, and the concrete fragments were weighed before and after drying to constant weight in an oven at 230ºF, which was achieved in 14 days in this research. The gravimetric moisture contents were then calculated for a set of cylinders that was cured for 28 days in the fog room and then placed on the laboratory bench in open air and a set of cylinders that was cured in the fog room for the entire 1 year for each of the four bridge decks.

3.7 Field Testing

Field testing involved Schmidt rebound hammer testing, chloride concentration testing, and distress surveys on each bridge deck as described in the following sections.

3.7.1 Schmidt Rebound Hammer Testing

Schmidt rebound hammer testing was performed to non-destructively estimate the in-situ compressive strength of the concrete at 1 year and 2 years following deck construction. In the test, which involves impacting the concrete surface with a spring-loaded hammer as shown in Figure 3-16, higher rebound numbers correspond to higher concrete compressive strengths. Seven locations on each bridge deck were evaluated. Three were positioned over each of the

Figure 3-16 Schmidt rebound hammer testing.

three sensor locations, as shown in Appendix A, and four were positioned as shown in Figures 3-17 and 3-18 for the Dannon Way and 8200 South bridges, respectively. Among these latter four locations, regardless of the direction of traffic on the deck, two were consistently located 25 and 75 ft from the north end of the west lane and two were consistently located 25 and 75 ft from the south end of the east lane along a line 5 ft from the inside face of the parapet wall in both cases. Three tests were conducted at each of the seven test locations on each deck in accordance with ASTM C805 (Standard Test Method for Rebound Number of Hardened Concrete). As required, the surface of the concrete at each location was smoothed with a grinding stone before the testing was performed.

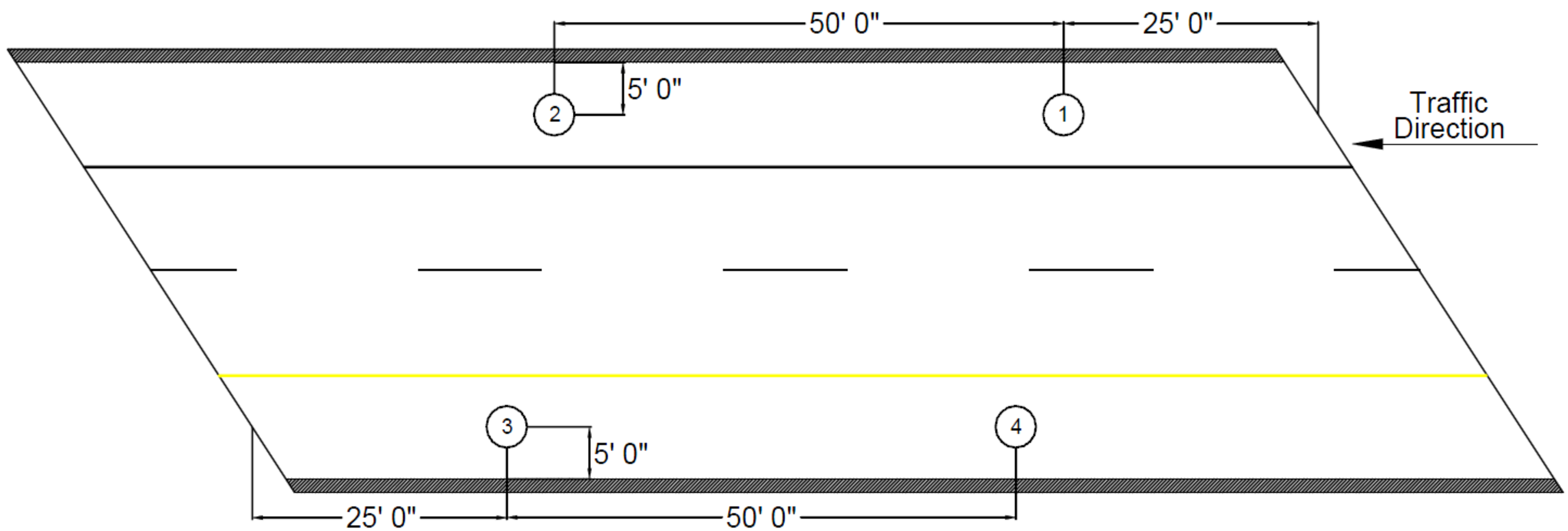

Figure 3-17 Testing locations in shoulders of Dannon Way bridges.

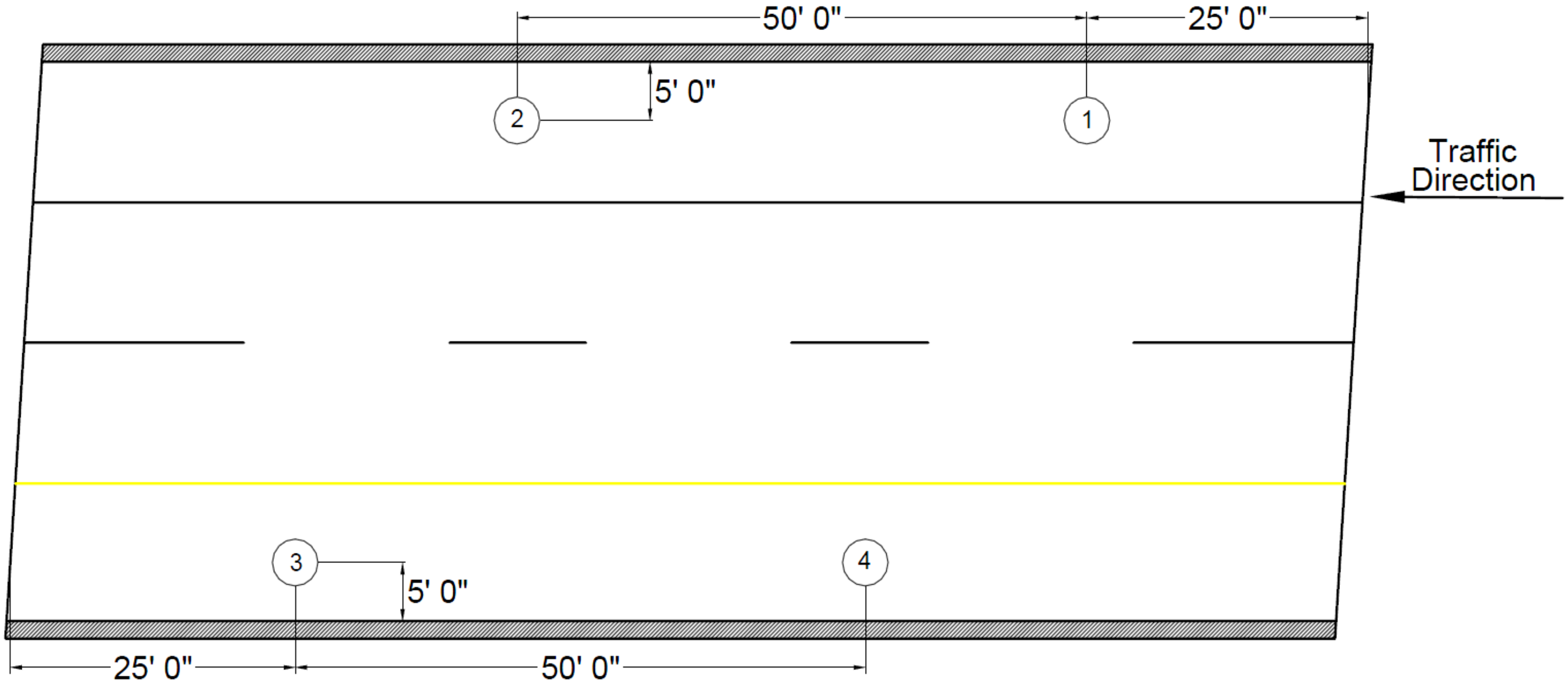

Figure 3-18 Testing locations in shoulders of 8200 South bridges.

3.7.2 Chloride Concentration Testing

Chloride concentration testing was conducted on each bridge deck at 1 year and 2 years following deck construction. In order to comply with Utah DOT requirements, testing was limited to only four locations, including two on the east shoulder and two on the west shoulder of each deck that coincided with the Schmidt rebound hammer test locations shown in Figures 3-17 and 3-18 for the Dannon Way and 8200 South bridges, respectively. At each location, a hole was drilled to a depth of 1 in. in two 0.5-in. lifts with a hammer drill as depicted in Figure 3-19; the holes were deliberately situated away from the underlying reinforcing steel, which was located using a cover meter. The shallower lift was drilled with a 1.5-in.-diameter bit, while the deeper lift was drilled with a 1.0-in.-diameter bit. Use of a smaller bit for the deeper lift prevented inadvertent scraping of near-surface concrete that would have otherwise contaminated the deeper sample. After each lift was drilled, the pulverized concrete powder was collected and bagged for titration at the BYU Highway Materials Laboratory. Chloride concentrations were

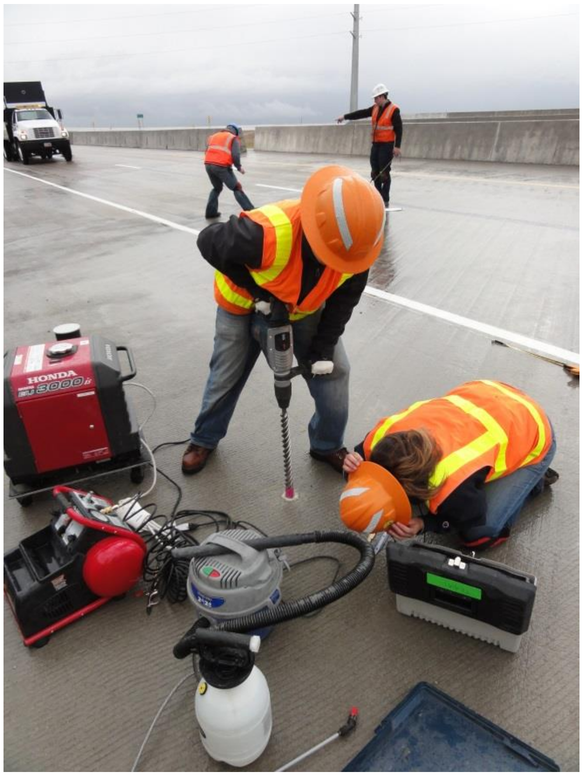

Figure 3-19 Concrete sampling for chloride concentration testing.

determined by weight per cubic yard of concrete using the theoretical concrete densities previously presented in Table 3-2.

3.7.3 Distress Surveys

Deck distress surveys were conducted to quantify and compare the degree of surface cracking among the bridge decks at approximately 2 months, 5 months, 8 months, 1 year, and 2 years following deck construction. Except for the 2-month survey, which occurred before the inspection protocols were formalized, the extent and severity of any deck surface cracking was documented in terms of crack lengths and widths, respectively, and the crack locations were also recorded. Distress surveys performed at 2, 5, and 8 months were conducted prior to the decks being opened to regular trafficking, which occurred immediately after the 8-month surveys were completed; the decks were exposed only to construction traffic before this time.

As suggested in Figures 3-16 and 3-19, the 1-year distress survey was conducted in rainy conditions for the first few hours of testing. The wet condition of the deck surface may have prevented identification of some cracking on the SB decks, which were tested first. However, the order of the inspections ensured that the effect of the rain was distributed as evenly as possible across both decks in that direction, as both right lanes were inspected before both left lanes. The same order of inspections was repeated on the NB decks but in the opposite direction. For the 2-year distress survey, conditions were dry, as shown in Figure 3-20, and the same order of inspections used for the 1-year testing was applied for consistency.

After all of the distress surveys were performed, electronic maps of the cracking were prepared in AutoCAD for each deck for each inspection time. Maps previously prepared for the 5-month, 8-month, and 1-year surveys (*23*, *24*, *25*) were revised, as needed, to better align with those prepared for the 2-year survey, which was performed using more rigorous methods of defining crack locations than had been previously employed. In addition, the total lengths of the cracking were calculated for all the maps using an automated computer algorithm within the AutoCAD software environment rather than the manual estimation method previously utilized (*23*, *24*, *25*).

Figure 3-20 Distress survey.

3.8 Summary

In the spring of 2012, the Mountain View Corridor project team of the Utah DOT constructed four new bridges in West Jordan, Utah, including two at Dannon Way and two at 8200 South. At each location, one bridge deck was constructed using a conventional concrete mixture, and one was constructed using a concrete mixture containing a portion of pre-wetted LWFA to facilitate internal curing. Each structure incorporated five pre-stressed, pre-cast concrete girders and pre-cast half-deck concrete panels placed between the girders. Prior to concrete placement, each of the four bridge decks was instrumented with three sensors connected to a data logger equipped with cellular telephone service. The sensors measured the volumetric moisture content, temperature, and electrical conductivity of the concrete on hourly intervals. Air temperature, relative humidity, and precipitation gauges were mounted on the bridges to monitor the ambient conditions to which the bridge decks were exposed.

The concrete bridge deck construction procedures were consistent for each bridge. The concrete placed on all four decks was supplied by the same concrete producer and placed and finished by the same construction crew. The concrete mixture designs were identical to each other except for the inclusion of the pre-wetted LWFA in the internally cured concrete, a 30 percent replacement of the fine aggregate by volume. For the internally cured concrete, the

LWFA was pre-wetted for a minimum of 2 days at the batch plant using a sprinkling system to achieve a minimum moisture content of 15 percent, and excess water was then allowed to drain prior to concrete batching.

Laboratory procedures were conducted to determine the compressive strength, chloride permeability, elastic modulus, and moisture content of the two concrete mixtures. During placement of each of the four bridge decks, 10 concrete cylinders were cast using concrete sampled from each of the three sensor locations, for a total of 30 concrete cylinders from each deck. Compressive strength testing was performed on one or two sets of cylinders from each bridge deck at 7 days, 28 days, 58 days, 3 months, 6 months, and 1 year following deck construction, where a set included one cylinder from each sensor location on a given deck. The RCPT was performed on one or two sets of cylinders from each bridge deck at 28 days, 6 months, and 1 year, and modulus tests and moisture content tests were conducted on two sets of cylinders from each bridge deck at 1 year.

Field testing consisted of Schmidt rebound hammer testing, chloride concentration testing, and distress surveys. Schmidt rebound hammer testing and chloride concentration testing were performed at seven and four locations, respectively, on each bridge deck at 1 and 2 years following deck construction. Deck distress surveys were conducted to quantify and compare the degree of surface cracking among the bridge decks at approximately 2 months, 5 months, 8 months, 1 year, and 2 years following deck construction.

4.0 RESULTS

4.1 Overview

The following sections provide results from sensor readings, laboratory testing, and field testing performed in this research.

4.2 Sensor Readings

Sensor readings through 2 years of moisture content, temperature, and electrical conductivity monitoring are presented in the following sections. A sample output file containing sensor data is provided in Appendix B.

4.2.1 Moisture Content and Temperature Measurements

Volumetric moisture contents, generally computed as the average of the readings obtained from each of the three sensors on each bridge deck at a given time, are shown in Figures 4-1 and 4-2 and summarized in Table 4-1. The average volumetric moisture content of the internally cured concrete was 3 to 4 percentage points higher after 7 days and 2 to 3 percentage points higher after 28 days, 3 months, 6 months, and 1 year than the volumetric moisture content of the conventional concrete. The higher moisture content of the internally cured concrete during this time can be attributed to the use of pre-wetted LWFA in the mixture, which provides an internal source of readily available water. At 2 years, however, the difference in average moisture content between the internally cured concrete and the conventional concrete was much smaller, decreasing to less than 2 percentage points. The converging moisture contents for the internally cured and conventional concrete decks can be attributed to the gradual release and chemical reaction and/or evaporation of the water from the pores within the LWFA during the curing process. (Attributable to cracking that occurred in the internally cured decks at some sensor locations, unrepresentative and possibly erroneous moisture content measurements were observed in the data after approximately 305 days from the date of construction. These measurements are not included in Figures 4-1 or 4-2 or Table 4-1. Graphs showing the complete data sets, without removal of the unrepresentative data, are provided in Appendix B.)

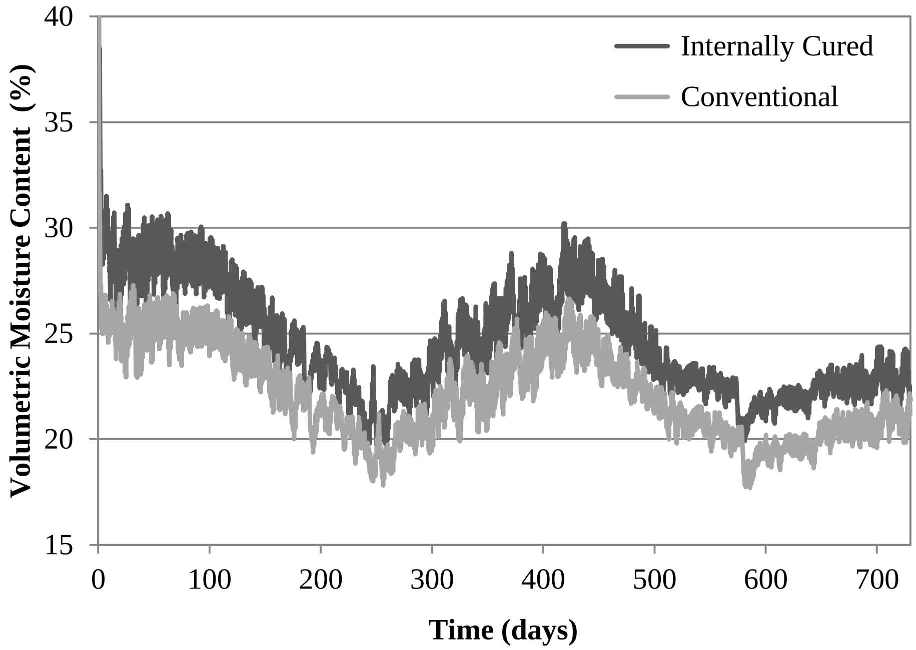

Figure 4-1 Average volumetric moisture content for decks located at Dannon Way.

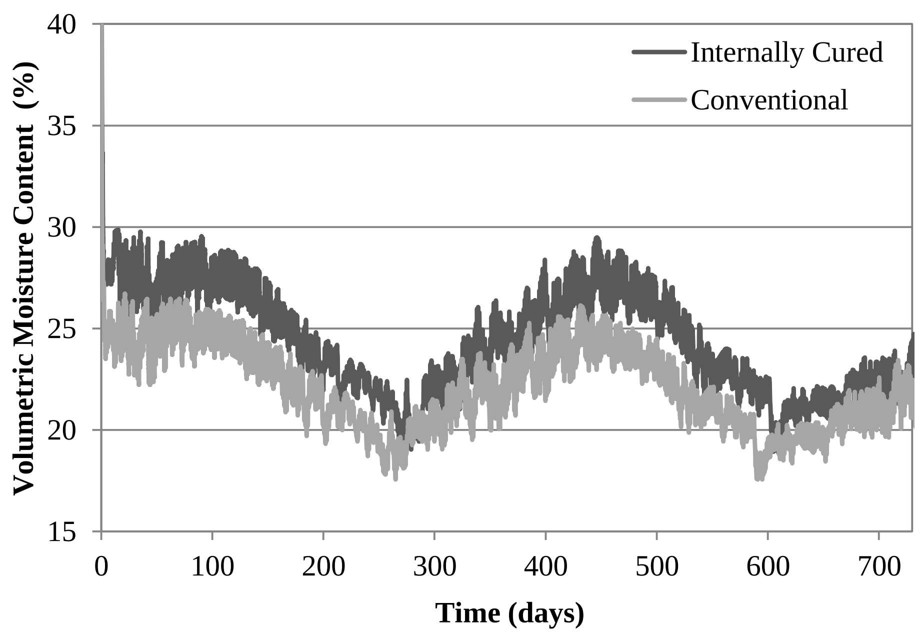

Figure 4-2 Average volumetric moisture content for decks located at 8200 South.

Table 4-1 Deck Volumetric Moisture Content

Location	Volumetric Moisture Content (%)											
	7-day		28-day		3-month		6-month		1-year		2-year	
	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.
Dannon Way SB Conventional	25.9	1.1	25.3	0.8	24.9	1.0	20.3	0.9	21.8	0.9	21.2	2.2
Dannon Way NB Internally Cured	29.9	1.0	28.6	0.7	27.5	0.4	23.5	0.7	24.5	1.0	21.8	0.0
8200 S NB Conventional	24.9	0.7	25.2	0.6	24.9	0.6	21.9	0.6	22.3	0.7	20.9	0.4
8200 S SB Internally Cured	27.9	1.0	27.3	0.7	26.6	0.6	22.3	0.8	24.0	0.7	22.8	2.0

Figures 4-3 and 4-4 show the internal concrete deck temperatures at Dannon Way and 8200 South, respectively, compared to the ambient temperatures. As the deck temperatures decreased below 32°F, the apparent moisture contents shown in Figures 4-1 and 4-2 also decreased. The reduction in water content is attributable to the fact that the in-situ sensors measure only liquid water, not ice; while water in the fine gel pores within the cement paste seldom freezes under typical service conditions, water in the larger capillary pores can change to ice at subfreezing temperatures and would then be undetected by the sensors (*26*). The frequent oscillations of temperature above and below 32°F and the corresponding oscillations in moisture content indicate the occurrence of freezing and thawing of the concrete bridge decks throughout the winter months.

The influence of relative humidity and precipitation on the measured in-situ volumetric moisture contents of the concrete bridge decks can be evaluated using the data shown in Figures 4-5 through 4-8. (The precipitation gauges were not installed until approximately 5 months after deck construction, so Figures 4-7 and 4-8 show data beginning at 150 days.) Although higher ambient relative humidity values correspond to lower water evaporation rates from the surface of concrete, the measured internal moisture contents are not apparently affected by changes in relative humidity. However, even with average cover depths over the embedded sensors ranging

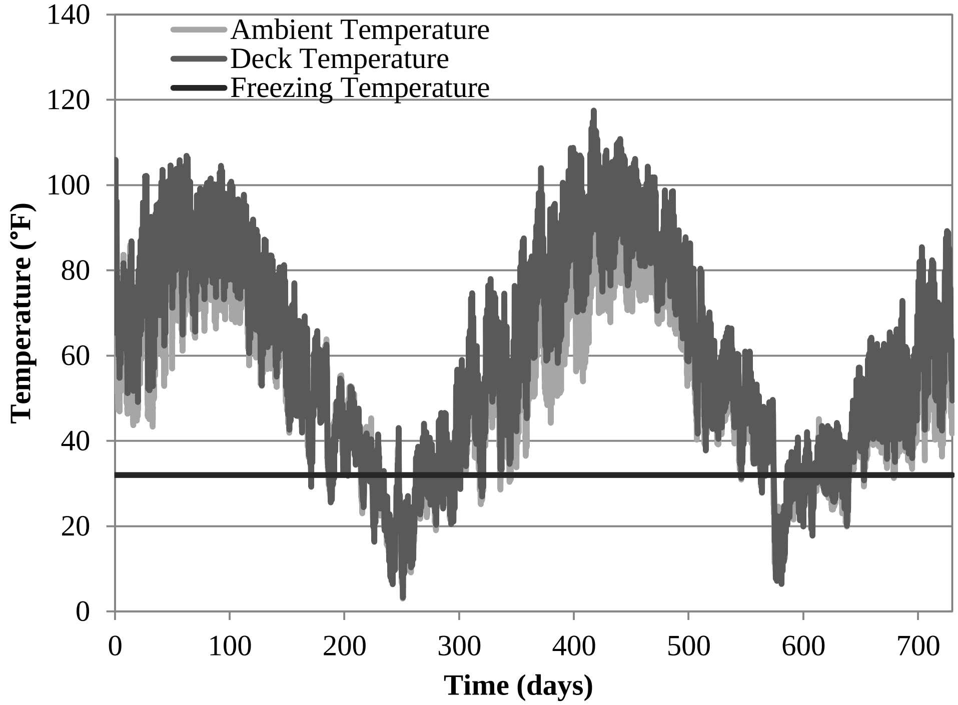

Figure 4-3 Average temperatures for decks located at Dannon Way.

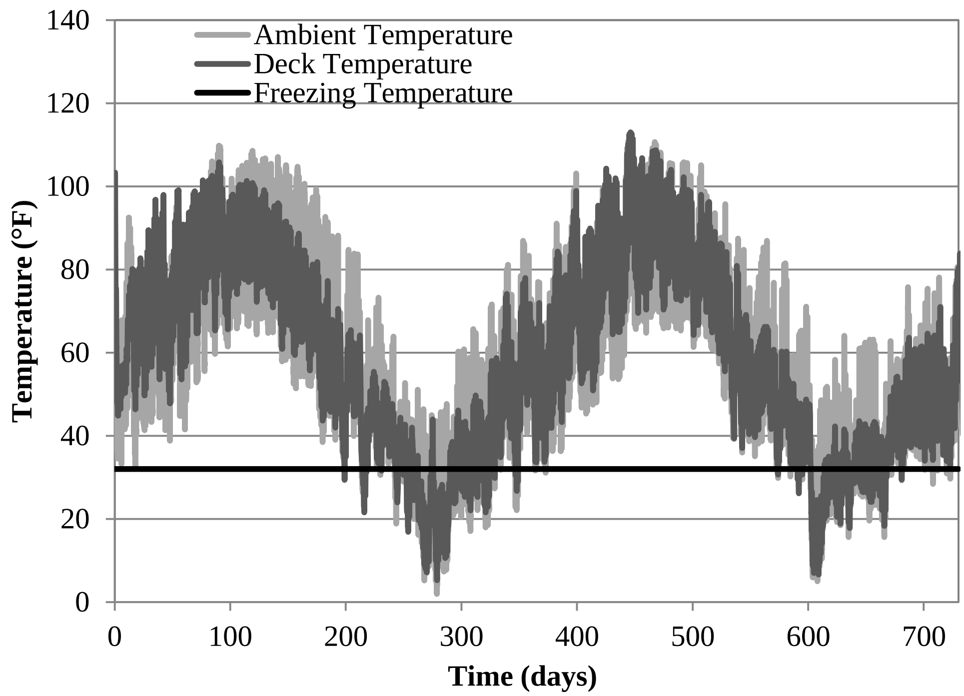

Figure 4-4 Average temperatures for decks located at 8200 South.

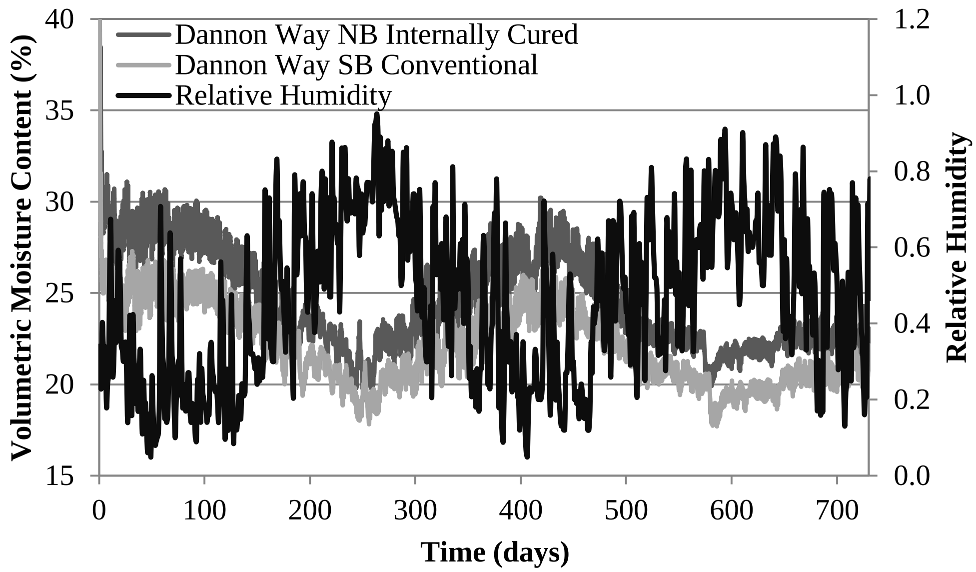

Figure 4-5 Volumetric moisture content and relative humidity for decks located at Dannon Way.

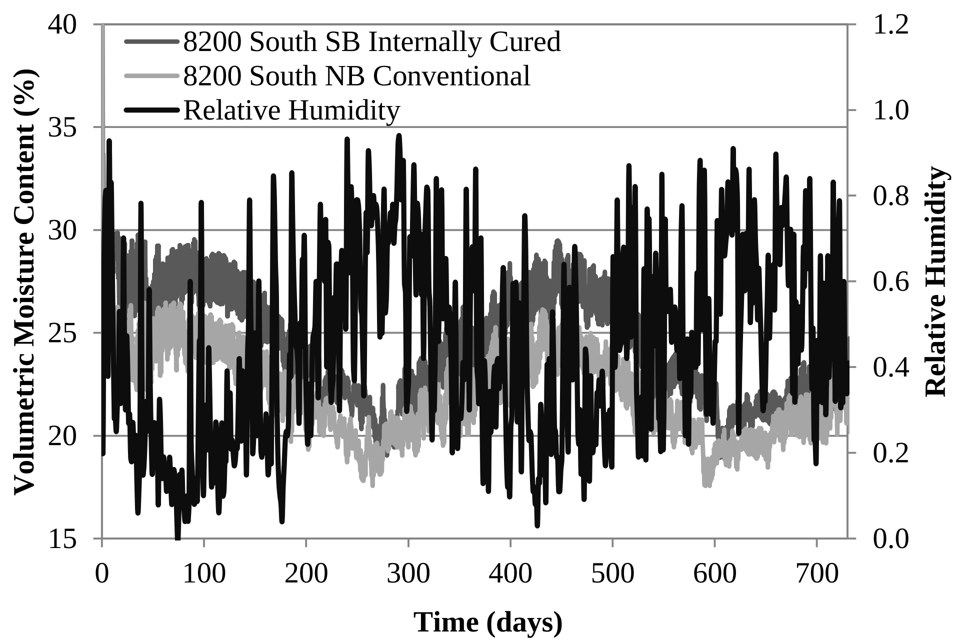

Figure 4-6 Volumetric moisture content and relative humidity for decks located at 8200 South.

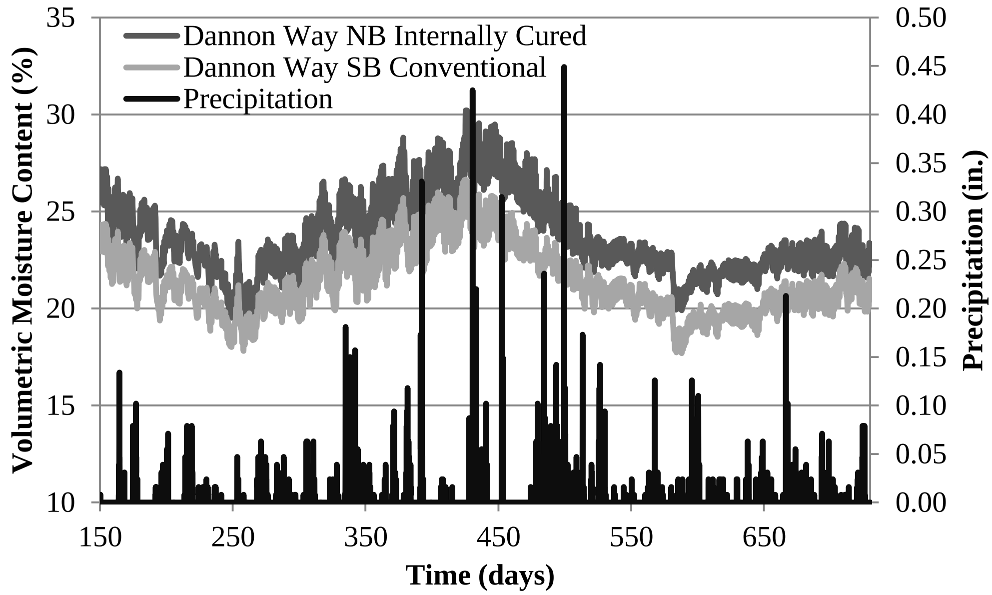

Figure 4-7 Volumetric moisture content and precipitation for decks located at Dannon Way.

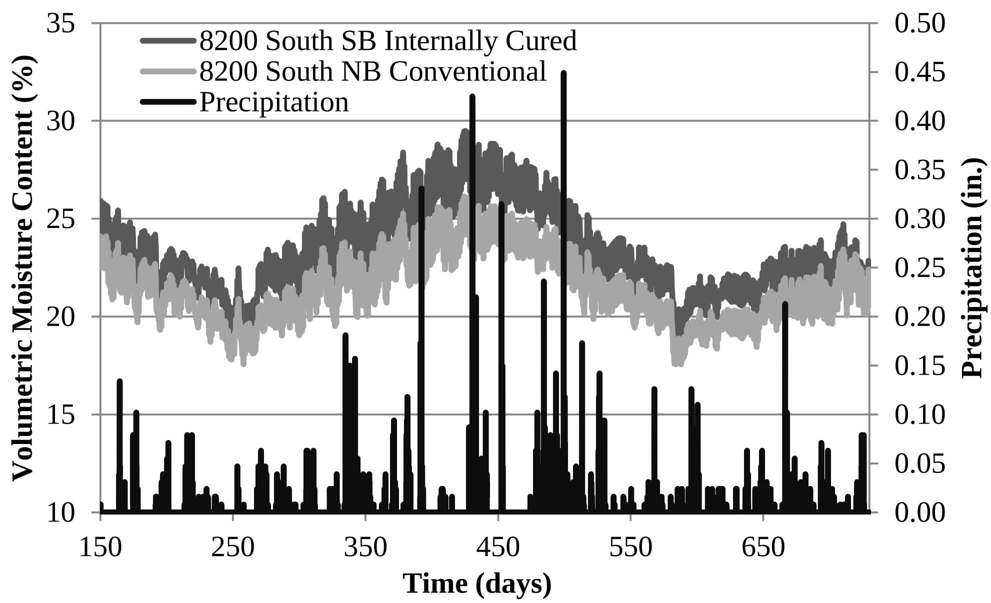

Figure 4-8 Volumetric moisture content and precipitation for decks located at 8200 South.

from 2.4 to 3.4 in. on these bridge decks, the moisture contents at the depth of the sensors do appear to be somewhat correlated with the occurrence and magnitude of precipitation events, at least on a seasonal basis.

4.2.2 Electrical Conductivity Measurements

The electrical conductivity of concrete serves as an indicator of concrete diffusivity. Diffusivity, concrete permeability, and the amount of concrete cover over reinforcing steel are all key factors in preventing the occurrence of corrosion (*27*). In general, the electrical conductivity values consistently decreased during the first 2 months and then began stabilizing, with only marginal changes over the subsequent months as shown in Figures 4-9 and 4-10 and summarized in Table 4-2.

Although the internally cured concrete decks had a consistently higher average moisture content, which may initially suggest a higher diffusivity, the average electrical conductivity values of the internally cured concrete decks were not consistently higher than those measured on the conventional concrete decks, especially during the first 3 months of the monitoring period. However, at 6 months, 1 year, and 2 years, the average electrical conductivity of the internally cured decks was 50, 38, and 50 percent greater than that of the conventional concrete decks, respectively. The higher electrical conductivity of the internally cured decks suggests that they are potentially developing greater susceptibility to chloride ingress than the conventional concrete decks. (Again, attributable to cracking that occurred in the internally cured decks at some sensor locations, unrepresentative and possibly erroneous electrical conductivity measurements were observed in the data after approximately 305 days from the date of construction. These measurements are not included in Figures 4-9 or 4-10 or Table 4-2. Graphs showing the complete data sets, without removal of the unrepresentative data, are provided in Appendix B.)

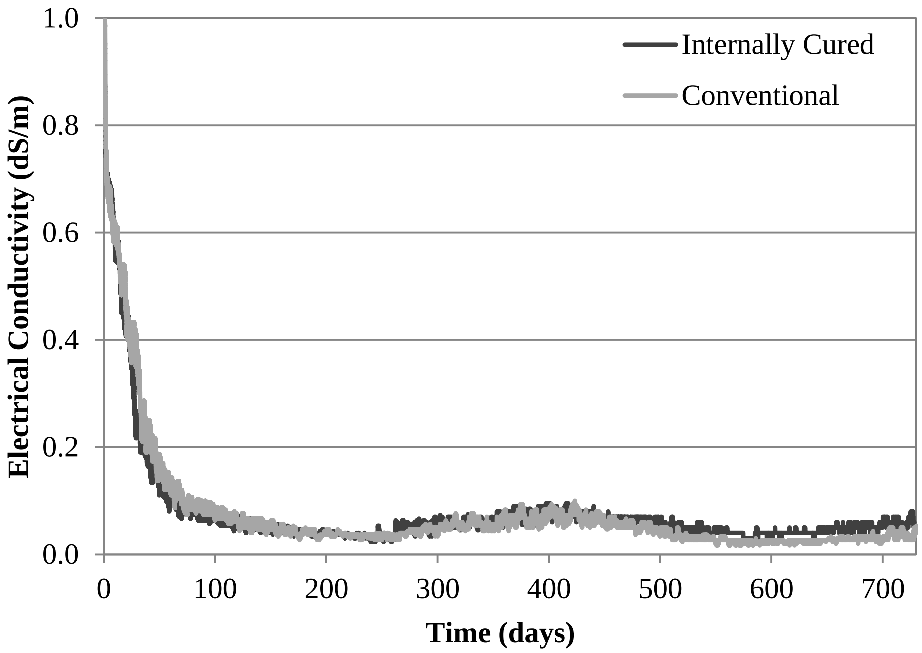

Figure 4-9 Average electrical conductivity for decks located at Dannon Way.

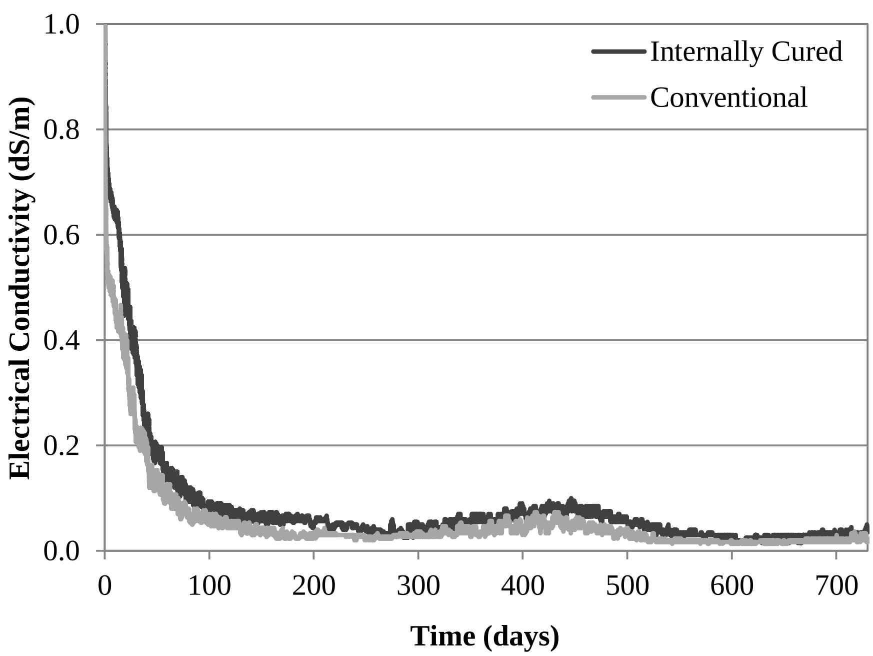

Figure 4-10 Average electrical conductivity for decks located at 8200 South.

Table 4-2 Deck Electrical Conductivity

Location	Electrical Conductivity (dS/m)											
	7-day		28-day		3-month		6-month		1-year		2-year	
	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.
Dannon Way SB Conventional	0.64	0.12	0.39	0.05	0.08	0.01	0.03	0.01	0.05	0.00	0.04	0.00
Dannon Way NB Internally Cured	0.66	0.07	0.27	0.03	0.07	0.01	0.04	0.00	0.06	0.00	0.05	0.00
8200 S NB Conventional	0.49	0.06	0.30	0.03	0.07	0.01	0.03	0.01	0.04	0.01	0.02	0.00
8200 S SB Internally Cured	0.66	0.04	0.40	0.02	0.07	0.01	0.05	0.03	0.06	0.00	0.04	0.00

4.3 Laboratory Testing

Laboratory testing was performed in parallel with the sensor readings for comparison of the conventional and internally cured concrete mixtures. The results of compressive strength testing, rapid chloride permeability testing, modulus testing, and moisture content testing are presented in the following sections.

4.3.1 Compressive Strength Testing

Concrete compressive strength is primarily influenced by concrete mixture proportions, curing conditions, and the age at which the sample is tested. Previous research indicates that internally cured concrete generally exhibits lower compressive strengths at ages less than 28 days and higher compressive strengths between 28 days and 3 months compared to conventional concrete (*28*, *29*). However, comparisons of strength data for conventional and internally cured concrete or mortar specimens tested beyond a curing time of 3 months were not identified in the literature review performed for this research.

Figure 4-11 and Table 4-3 show the 7-day, 28-day, 3-month, 58-day, 6-month, and 1-year compressive strengths measured for each bridge deck, and measurements for individual specimens are given in Appendix B. (In Table 4-3, a hyphen means that the standard deviation could not be computed because only one specimen was tested in each case. Valid data for additional specimens was not obtained due to equipment errors.) At 7 days, the conventional concrete was stronger by an average of nearly 4.5 percent, or about 200 psi, than the internally cured concrete; however, at 28 days, the internally cured concrete was stronger by an average of nearly 1.0 percent, or about 50 psi, than the conventional concrete. As shown in Appendix B, these compressive strengths measured at BYU are consistent with those obtained by the Utah DOT for quality assurance purposes. At 3 months, the internally cured concrete was still stronger by an average of 1.0 percent, or about 60 psi, than the conventional concrete, but at 6 months the conventional concrete was stronger by 1.5 percent, or nearly 100 psi. Thus, for the first 6 months, the two concrete mixtures exhibited very similar strength characteristics. However, at 1 year, greater variability was observed in that the conventional concrete was stronger by an average of 12.9 percent, or nearly 900 psi, than the internally cured concrete. For the cylinders that were cured in the fog room for the entire 1 year, the conventional concrete was 7.4 percent, or about 450 psi, stronger than the internally cured concrete.

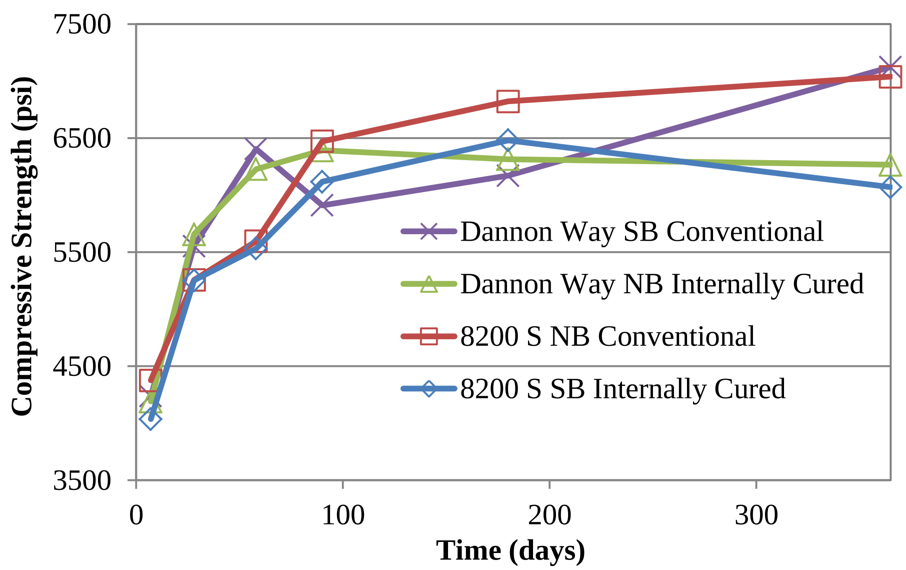

Figure 4-11 Average concrete compressive strength.

Table 4-3 Concrete Compressive Strength

Bridge Location	Compressive Strength (psi)													
	7-day		28-day		58-day*		3-month		6-month		1-year		1-year (Fog Room Only)	
	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.
Dannon Way SB Conventional	4238	270	5552	417	6404	459	5910	1491	6170	695	7121	484	6097	655
Dannon Way NB Internally Cured	4191	112	5656	592	6227	491	6393	897	6315	-	6266	475	5806	520
8200 South NB Conventional	4377	112	5256	797	5596	851	6472	202	6821	515	7038	745	6953	353
8200 South SB Internally Cured	4036	173	5252	506	5532	543	6117	250	6481	-	6069	788	6340	354

* Compressive strength testing following 30 days of freeze-thaw cycling

Because of the high number of freeze-thaw cycles experienced by concrete bridge decks every year in Utah, freeze-thaw damage is a major concern. The compressive strengths of the two concrete mixtures shown at 58 days in Figure 4-11 were measured after 30 freeze-thaw cycles were applied in the laboratory following 28 days of curing in the fog room. The freeze-thaw cycling had no evident impact on the strength gain characteristics of either type of concrete, although the conventional concrete was stronger by 2.0 percent, or about 120 psi, than the internally cured concrete following the freeze-thaw cycling. These results are consistent with those obtained in previous freeze-thaw tests, in which the inclusion of lightweight aggregate neither improved nor worsened the freeze-thaw performance of the concrete (*30*); as in the current study, the low water-cementitious materials ratio characteristic of the specimens tested in that research may have prevented the higher water content typical of internally cured concrete from having an adverse impact on freeze-thaw durability.

The decreased compressive strengths of the internally cured concrete compared to the conventional concrete tested at 1 year in this research are similar to the results of a study performed to investigate the influence of super-absorbent polymers (SAPs) on mortar properties (*31*). In that study, reductions in compressive strength of up to 10 percent were observed at 28 days for mixtures containing between 0.1 and 0.6 percent SAPs by mass of cement. Similar to the effect of SAPs, while additional available water located in the permeable pores of the LWFA can increase the degree of cement hydration, decreases in the strength of internally cured concrete over time compared to conventional concrete may be associated with increased void contents in the concrete, particularly in concrete mixtures with water-cementitious materials ratios above 0.45 (*31*). Alternatively, the LWFA could be mechanically weaker than the normal-weight aggregate it replaced (*7*).

4.3.2 Rapid Chloride Permeability Testing

The addition of LWFA to promote internal curing has been shown to increase the degree of cement hydration and densify the concrete microstructure and interfacial transition zone, which should in turn increase the resistance of the concrete to chloride penetration (*30*). Figure

4-12 and Table 4-4 show the 28-day, 6-month, and 1-year RCPT results obtained for each bridge deck, and measurements for individual specimens are given in Appendix B.

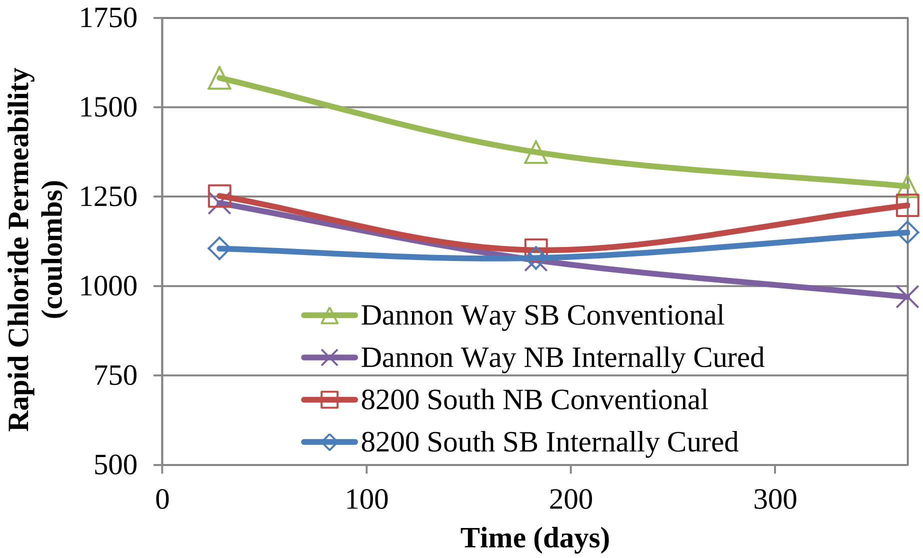

Figure 4-12 Average rapid chloride permeability.

Table 4-4 Rapid Chloride Permeability

Bridge Location	Rapid Chloride Permeability (coulombs)							
	28-day		6-month		1-year		1-year (Fog Room Only)	
	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.
Dannon Way SB Conventional	1582	399	1374	102	1279	275	397	21
Dannon Way NB Internally Cured	1232	108	1072	217	969	166	363	31
8200 South NB Conventional	1252	102	1100	48	1226	382	354	23
8200 South SB Internally Cured	1105	44	1078	240	1150	222	379	25

In the RCPT, the internally cured concrete passed 17.5 percent less current at 28 days, 13.1 percent less current at 6 months, and 15.4 percent less current at 1 year than that passed by the conventional concrete at the same curing times, on average, when the cylinders were cured for 28 days in the fog room and then placed on the laboratory bench in open air for the remaining time. However, despite the numerical differences in results between the two types of concrete, they are both classified as having low chloride permeability at all of these time intervals; the only exception is the internally cured concrete from the Dannon Way NB deck, which is classified as having very low chloride permeability at 1 year.

For the cylinders that were cured in the fog room for the entire 1 year, the internally cured concrete passed 1.3 percent less current than that passed by the conventional concrete, on average. However, the measured chloride permeability was much lower for both types of concrete; on average, the cylinders cured in the fog room passed 70.0 percent and 65.0 percent less current at 1 year than that passed by the cylinders cured on the laboratory bench in open air for the same time for conventional and internally cured concrete, respectively. All cylinders cured in the fog room are classified as having very low chloride permeability in this case.

4.3.3 Modulus Testing

Table 4-5 presents the elastic modulus values measured at 1 year for a set of cylinders that was cured for 28 days in the fog room and then placed on the laboratory bench in open air and a set that was cured in the fog room for the entire 1 year for each bridge deck, and measurements for individual specimens are given in Appendix B. For the cylinders cured in open air after 28 days, the modulus of the internally cured concrete was 3.9 percent lower, on average, than that of the conventional concrete, which is consistent with previous research (*32*). A reduction in elastic modulus has been shown to be correlated with a reduction in cracking potential (*33*); reductions in stiffness associated with lower modulus values can reduce stresses that lead to cracking by 10 to 20 percent (*29*).

Table 4-5 Modulus of Elasticity

Bridge Location	Modulus of Elasticity (10^6 psi)			
	Open Air		Fog Room Only	
	Avg.	St. Dev.	Avg.	St. Dev.
Dannon Way SB Conventional	5.00	0.13	4.68	0.04
Dannon Way NB Internally Cured	4.94	0.16	5.64	0.18
8200 South NB Conventional	5.00	0.19	4.44	0.07
8200 South SB Internally Cured	4.66	0.21	5.11	0.01

For the cylinders that were cured in the fog room for the entire 1 year, the opposite trend was observed. The modulus of the internally cured concrete was 17.9 percent higher, on average, than that of the conventional concrete; the modulus of the internally cured concrete increased as a result of continuous curing in the fog room, while the modulus of the conventional concrete decreased. In all cases, however, the measured modulus values were within normal ranges for concrete.

4.3.4 Moisture Content Testing

Table 4-6 presents the gravimetric moisture contents measured at 1 year for a set of cylinders that was cured for 28 days in the fog room and then placed on the laboratory bench in open air and a set that was cured in the fog room for the entire 1 year for each bridge deck, and measurements for individual specimens are given in Appendix B. For the cylinders cured in open air after 28 days, the moisture content of the internally cured concrete was 0.5 percentage points higher, on average, than that of the conventional concrete. For the cylinders that were cured in the fog room for the entire 1 year, where an ample supply of external water was available, the moisture content of the internally cured concrete was in this case 2 to 3 percentage points higher, on average, than that of the conventional concrete.

Table 4-6 Gravimetric Moisture Content

Location	Gravimetric Moisture Content (%)			
	Open Air		Fog Room Only	
	Avg.	St. Dev.	Avg.	St. Dev.
Dannon Way SB Conventional	2.42%	0.06%	6.25%	0.33%
Dannon Way NB Internally Cured	3.10%	0.22%	8.36%	0.25%
8200 S NB Conventional	2.88%	0.28%	6.59%	0.30%
8200 S SB Internally Cured	3.39%	0.09%	8.91%	0.17%

4.4 Field Testing

The results of Schmidt rebound hammer testing, chloride concentration testing, and distress surveys performed on each bridge deck are presented in the following sections.

4.4.1 Schmidt Rebound Hammer Testing

Table 4-7 provides in-situ compressive strengths of the concrete as estimated using the Schmidt rebound hammer for each bridge deck; Schmidt rebound numbers and estimated compressive strengths for individual test locations are given in Appendix B. The internally

Table 4-7 Concrete Compressive Strength from Schmidt Rebound Hammer Testing

Bridge Location	Compressive Strength (psi)			
	1-year		2-year	
	Avg.	St. Dev.	Avg.	St. Dev.
Dannon Way SB Conventional	7200	522	7543	336
Dannon Way NB Internally Cured	6950	418	7214	474
8200 S NB Conventional	6764	839	7600	265
8200 S SB Internally Cured	7007	566	7507	354

cured concrete was neither consistently stronger nor weaker than the conventional concrete after 1 year. However, the internally cured concrete was weaker than the conventional concrete at 2 years following deck construction, which is more consistent with the trend observed in the laboratory testing at 1 year.

4.4.2 Chloride Concentration Testing

Table 4-8 provides chloride concentrations for the upper 1 in. of the deck surface for each of the bridge decks. On average, the internally cured concrete exhibited 1.4 percent greater chloride concentration in the 0.0 to 0.5 in. depth interval and 15.7 percent greater chloride concentration in the 0.5 to 1.0 in. depth interval than the conventional concrete at 1 year following deck construction. At 2 years following deck construction, the internally cured concrete exhibited 12.3 percent greater chloride concentration in the 0.0 to 0.5 in. depth interval and 46.2 percent greater chloride concentration in the 0.5 to 1.0 in. depth interval than the conventional concrete, on average. Although contrary to the results of the RCPTs performed in the laboratory, these differences are consistent with the higher electrical conductivity values previously reported for the internally cured concrete.

The comparatively high chloride concentrations measured for both types of concrete after just 2 years are evidence of the high amounts of deicers that are distributed on Utah bridge decks during winter. Substantial cracking, which is discussed in the next section, is likely also a significant contributor to the comparatively high level of chlorides observed. Even though the

Table 4-8 Chloride Concentration

Bridge Location	Depth (in.)	Chloride Concentration (lb Cl^-/yd^3 Concrete)			
		1-year		2-year	
		Avg.	St. Dev.	Avg.	St. Dev.
Dannon Way SB Conventional	0.0 to 0.5	8.34	2.98	10.72	1.54
	0.5 to 1.0	1.91	1.26	1.96	2.19
Dannon Way NB Internally Cured	0.0 to 0.5	8.09	1.35	11.20	0.83
	0.5 to 1.0	1.85	1.05	4.11	2.21
8200 South NB Conventional	0.0 to 0.5	8.57	2.34	10.78	2.44
	0.5 to 1.0	1.59	0.78	1.59	0.82
8200 South SB Internally Cured	0.0 to 0.5	9.05	3.70	13.33	1.95
	0.5 to 1.0	2.20	1.38	2.49	1.40

measured cover depths are generally satisfactory, ranging from 2.4 to 3.4 in. on these decks, cracking provides a direct pathway for the penetration of chloride ions, increasing the rate at which critical concentrations of chlorides can accumulate in the vicinity of the top mat of reinforcing steel (*11*). For this reason, Utah DOT engineers may wish to apply a surface treatment to each of the bridge decks as soon as possible upon completion of this research.

4.4.3 Distress Surveys

During the initial distress survey conducted at 2 months, three to five cracks were found on each of the conventional concrete bridge decks; a typical crack is shown in Figure 4-13. The cracks varied in length, were 0.005 to 0.010 in. in width, and were typically located on the north and south ends of the decks and at the location of the metal shoring, previously depicted in Figure 3-11, that was used in the construction process. However, at that time, no visible signs of cracking were found on the internally cured concrete bridge decks.

Table 4-9 shows the results of the distress surveys performed at 5 months, 8 months, 1 year, and 2 years, while Appendix C provides pictorial evidence of typical crack lengths and widths and also provides full distress maps prepared for each deck upon completion of these inspections. At 5 months, the average total crack lengths for the conventional and internally

Figure 4-13 Cracking in Dannon Way southbound conventional concrete deck at 2 months.

Table 4-9 Deck Cracking

Location	Total Cracking Length (in.)			
	5-month	8-month	1-year	2-year
Dannon Way SB Conventional	175	1045	6353	11953
Dannon Way NB Internally Cured	24	463	7106	18280
8200 South NB Conventional	257	3442	16774	25143
8200 South SB Internally Cured	66	221	2231	9352

cured concrete bridge decks were 216 and 45 in., respectively. At 8 months, following significant drops in temperature as winter commenced, the average total crack lengths increased to 2,244 and 342 in., respectively. Thus, on average, at 5 and 8 months, the conventional concrete bridge decks had 4.8 and 6.6 times more cracking, respectively, than the internally cured concrete decks. By 8 months, cracks in the conventional concrete bridge decks were well distributed throughout both bridge decks, while cracks in the internally cured concrete bridge decks were, with two exceptions, located at the north and south ends of the bridge where the approach slab meets the bridge deck and at the location where the metal shoring was used during the construction process. Cracks were typically 0.005 to 0.016 in. in width for both deck types at 5 and 8 months.

During the 1-year and 2-year distress surveys, significant increases in cracking were observed on each of the four bridge decks. At 1 year, the average total crack lengths for the conventional and internally cured concrete decks increased to 11,564 and 4,669 in., respectively, and, at 2 years, the average total crack lengths further increased to 18,548 and 13,816 in., respectively. Thus, on average, at 1 and 2 years, the conventional concrete decks had an average of 2.5 and 1.3 times more cracking, respectively, than the internally cured concrete decks. As exemplified in Figure 4-14, very distinctive reflection cracks from the joints between the underlying pre-cast half-deck panels were observed on all of the decks. The 8-ft longitudinal spacing between the reflection cracks along the majority of the length of the decks exactly matched the longitudinal dimensions of the panels, and the transverse offsets in the cracking patterns also matched the transverse offsets in the joint placements in adjacent rows of panels.

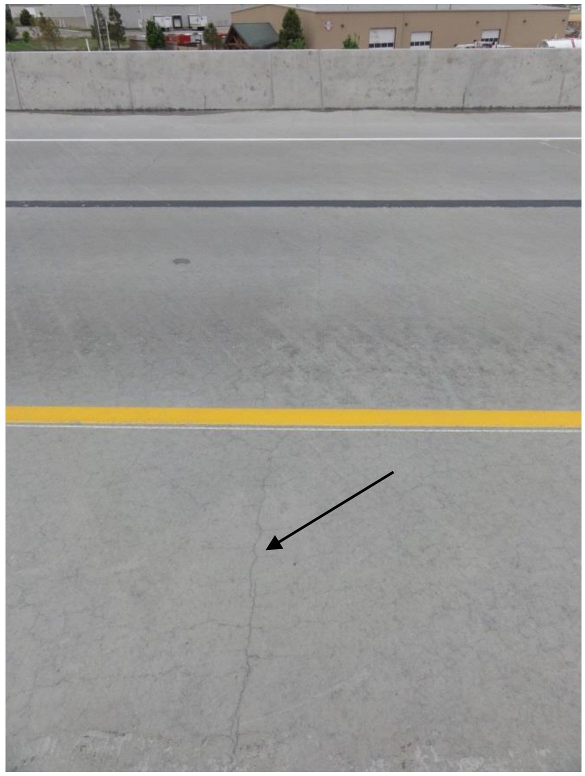

Figure 4-14 Reflection cracking from half-deck concrete panels in Dannon Way northbound internally cured concrete deck at 1 year.

As shown in Figure 4-15, cracks were typically 0.005 to 0.035 in. in width for both deck types at 1 year; however, at 2 years, crack widths ranged from 0.008 to 0.050 in., with most cracks being from 0.01 to 0.02 in. Furthermore, map cracking was also observed at several locations on both the conventional and internally cured concrete bridge decks, as indicated in Appendix C.

While the use of internally cured concrete did not prevent the occurrence of reflection cracking, it did apparently delay the propagation of such cracking. For example, at 8 months, the conventional concrete decks were starting to show evidence of transverse reflection cracking, particularly on the 8200 South NB deck, but none was observed on the internally cured concrete decks. In addition, at 1 year, the internally cured concrete decks exhibited significantly less offshoot cracking stemming from the transverse reflection cracking; indeed, in most cases, the reflection cracking on the internally cured concrete decks had little to no offshoot cracking, while the conventional concrete decks had significant offshoots creating a web-like pattern.

Figure 4-15 Cracking in 8200 South northbound conventional concrete deck at 1 year.

While thermal contraction of the half-deck panels during cooling assuredly played a role in the reflection cracking, trafficking probably also played a role. As previously explained, the panels were not connected across the transverse butt joints between them on any of the decks; thus, the top map of reinforcing steel was the only structural element providing continuous longitudinal support along the length of the deck. Under passing wheel loads, possible movement of the transverse joints between panels could have markedly increased the rate of crack formation.

These findings are consistent with several reports indicating that the use of pre-cast half-deck panels has led to transverse cracking in concrete bridge decks, where the cracks in the cast-in-place deck surface correspond with the butt joints between adjacent underlying panels. Although these cracks are not believed to significantly affect the structural performance of the deck (*34, 35*), such cracking may accelerate deck deterioration by allowing moisture and chloride ions to penetrate the concrete and initiate corrosion of the embedded reinforcing steel (*36*).

4.5 Summary

The average volumetric moisture content of the internally cured concrete was 3 to 4 percentage points higher after 7 days and 2 to 3 percentage points higher after 28 days, 3 months, 6 months, and 1 year following deck construction than the volumetric moisture content of the conventional concrete. At 2 years, however, the difference in average moisture content between the internally cured concrete and the conventional concrete was much smaller, decreasing to less than 2 percentage points. Although higher ambient relative humidity values correspond to lower water evaporation rates from the surface of concrete, the measured internal moisture contents are not apparently affected by changes in relative humidity. However, even with average cover depths over the embedded sensors ranging from 2.4 to 3.4 in. on these bridge decks, the moisture contents at the depth of the sensors do appear to be somewhat correlated with the occurrence and magnitude of precipitation events, at least on a seasonal basis. Although the internally cured concrete decks had a consistently higher average moisture content, which may initially suggest a higher diffusivity, the average electrical conductivity values of the internally cured concrete decks were not consistently higher than those measured on the conventional concrete decks, especially during the first 3 months of the monitoring period. However, at 6 months, 1 year, and 2 years, the average electrical conductivity of the internally cured decks was 50, 38, and 50 percent greater than that of the conventional concrete decks, respectively. The higher electrical conductivity of the internally cured decks suggests that they are potentially developing greater susceptibility to chloride ingress than the conventional concrete decks.

Laboratory compressive strength testing and rapid chloride permeability testing were performed in parallel with the sensor readings for comparison of the conventional and internally cured concrete mixtures. At 7 days following deck construction, the conventional concrete was stronger by an average of nearly 4.5 percent, or about 200 psi, than the internally cured concrete; however, at 28 days, the internally cured concrete was stronger by an average of nearly 1.0 percent, or about 50 psi, than the conventional concrete. At 3 months, the internally cured concrete was still stronger by an average of 1.0 percent, or about 60 psi, than the conventional concrete, but at 6 months the conventional concrete was stronger by 1.5 percent, or nearly 100 psi. Thus, for the first 6 months, the two concrete mixtures exhibited very similar strength characteristics. However, at 1 year, greater variability was observed in that the conventional concrete was stronger by an average of 12.9 percent, or nearly 900 psi, than the internally cured concrete. In the RCPT, the internally cured concrete passed 17.5 percent less current at 28 days, 13.1 percent less current at 6 months, and 15.4 percent less current at 1 year than that passed by the conventional concrete at the same curing times, on average. However, despite the numerical differences in results between the two types of concrete, with one exception, they are both classified as having low chloride permeability at all of these time intervals. In addition, laboratory free-free resonant testing at 1 year showed that the modulus of the internally cured concrete was 3.9 percent lower, on average, than that of the conventional concrete. For the tested specimens, the gravimetric moisture content of the internally cured concrete was 0.5 percentage points higher, on average, than that of the conventional concrete.

Field testing involved Schmidt rebound hammer testing, chloride concentration testing, and distress surveys on each bridge deck. The Schmidt rebound hammer testing at 1 year following deck construction showed that the internally cured concrete was neither consistently stronger nor weaker than the conventional concrete. However, at 2 years, the test showed that the internally cured concrete was weaker than the conventional concrete, which is more consistent with the trend observed in the laboratory testing at 1 year. On average, the internally cured concrete exhibited 1.4 percent greater chloride concentration in the 0.0 to 0.5 in. depth interval and 15.7 percent greater chloride concentration in the 0.5 to 1.0 in. depth interval than the conventional concrete at 1 year. At 2 years, the internally cured concrete exhibited 12.3 percent greater chloride concentration in the 0.0 to 0.5 in. depth interval and 46.2 percent greater chloride concentration in the 0.5 to 1.0 in. depth interval than the conventional concrete, on average. Although contrary to the results of the RCPTs performed in the laboratory, these differences are consistent with the higher electrical conductivity values previously reported for the internally cured concrete. On average, at 5 months, 8 months, 1 year, and 2 years following deck construction, the conventional concrete bridge decks had 4.8, 6.6, 2.5 and 1.3 times more cracking, respectively, than the internally cured concrete decks. During the 1-year and 2-year distress surveys, very distinctive reflection cracks from the joints between the underlying pre-cast half-deck panels were observed on all of the decks. The 8-ft longitudinal spacing between the cracks along the majority of the length of the decks exactly matched the longitudinal dimensions of the panels, and the transverse offsets in the cracking patterns also matched the transverse offsets in the joint placements in adjacent rows of panels. While the use of internally cured concrete did not prevent the occurrence of reflection cracking, it did apparently delay the propagation of such cracking. Although these cracks are not believed to significantly affect the structural performance of the deck, such cracking may accelerate deck deterioration by allowing moisture and chloride ions to penetrate the concrete and initiate corrosion of the embedded reinforcing steel.

5.0 CONCLUSION

5.1 Summary

The objectives of this research were to 1) monitor in-situ moisture and diffusivity for both conventional concrete and concrete containing pre-wetted LWFA, 2) compare deck performance in terms of early-age cracking, compressive strength, and chloride ingress, and 3) compare concrete properties in terms of compressive strength, chloride permeability, elastic modulus, and water content in the laboratory using cylinders cast in the field at the time of deck construction. The research involved field and laboratory evaluations of four newly constructed bridge decks located in the Mountain View Corridor in West Jordan, Utah. Two were constructed using conventional concrete, and two were constructed using pre-wetted LWFA to promote internal curing. Each structure incorporated five pre-stressed, pre-cast concrete girders and pre-cast half-deck concrete panels placed between the girders.

Prior to concrete placement, each of the four bridge decks was instrumented with moisture content, temperature, and electrical conductivity sensors connected to a data logger equipped with cellular telephone service. Air temperature, relative humidity, and precipitation gauges were also mounted on the bridges to monitor the ambient conditions to which the bridge decks were exposed.

During placement of the four bridge decks, concrete cylinders were cast using concrete sampled from each of the sensor locations, and laboratory compressive strength testing was then performed on cylinders from each bridge deck at 7 days, 28 days, 58 days, 3 months, 6 months, and 1 year following deck construction. In addition, rapid chloride permeability testing was performed on cylinders from each bridge deck at 28 days, 6 months, and 1 year, and modulus tests and moisture content tests were conducted on cylinders from each bridge deck at 1 year.

Field testing consisted of Schmidt rebound hammer testing, chloride concentration testing, and distress surveys. Schmidt rebound hammer testing and chloride concentration testing were performed on each bridge deck at 1 and 2 years following deck construction. Deck distress surveys were conducted to quantify and compare the degree of surface cracking among the bridge decks at approximately 2 months, 5 months, 8 months, 1 year, and 2 years following deck construction.

5.2 Findings

The average volumetric moisture content of the internally cured concrete was 3 to 4 percentage points higher after 7 days and 2 to 3 percentage points higher after 28 days, 3 months, 6 months, and 1 year following deck construction than the volumetric moisture content of the conventional concrete. At 2 years, however, the difference in average moisture content between the internally cured concrete and the conventional concrete was much smaller, decreasing to less than 2 percentage points. Although higher ambient relative humidity values correspond to lower water evaporation rates from the surface of concrete, the measured internal moisture contents are not apparently affected by changes in relative humidity. However, even with average cover depths over the embedded sensors ranging from 2.4 to 3.4 in. on these bridge decks, the moisture contents at the depth of the sensors do appear to be somewhat correlated with the occurrence and magnitude of precipitation events, at least on a seasonal basis. Although the internally cured concrete decks had a consistently higher average moisture content, which may initially suggest a higher diffusivity, the average electrical conductivity values of the internally cured concrete decks were not consistently higher than those measured on the conventional concrete decks, especially during the first 3 months of the monitoring period. However, at 6 months, 1 year, and 2 years, the average electrical conductivity of the internally cured decks was 50, 38, and 50 percent greater than that of the conventional concrete decks, respectively. The higher electrical conductivity of the internally cured decks suggests that they are potentially developing greater susceptibility to chloride ingress than the conventional concrete decks.

Laboratory compressive strength data indicate that, for the first 6 months following deck construction, the two concrete mixtures exhibited very similar strength gain characteristics. However, at 1 year, greater variability was observed in that the conventional concrete was stronger by an average of 12.9 percent, or nearly 900 psi, than the internally cured concrete. In the RCPT, the internally cured concrete consistently passed between 13.1 and 17.5 percent less current than that passed by the conventional concrete. However, despite the numerical differences in results between the two types of concrete, with one exception, they are both classified as having low chloride permeability. Laboratory free-free resonant testing at 1 year showed that the modulus of the internally cured concrete was 3.9 percent lower, on average, than that of the conventional concrete. For the tested specimens, the gravimetric moisture content of the internally cured concrete was 0.5 percentage points higher, on average, than that of the conventional concrete.

Field testing involved Schmidt rebound hammer testing, chloride concentration testing, and distress surveys on each bridge deck. The Schmidt rebound hammer testing at 1 year following deck construction showed that the internally cured concrete was neither consistently stronger nor weaker than the conventional concrete. However, at 2 years, the test showed that the internally cured concrete was weaker than the conventional concrete, which is more consistent with the trend observed in the laboratory testing at 1 year. On average, the internally cured concrete exhibited 1.4 percent greater chloride concentration in the 0.0 to 0.5 in. depth interval and 15.7 percent greater chloride concentration in the 0.5 to 1.0 in. depth interval than the conventional concrete at 1 year. At 2 years, the internally cured concrete exhibited 12.3 percent greater chloride concentration in the 0.0 to 0.5 in. depth interval and 46.2 percent greater chloride concentration in the 0.5 to 1.0 in. depth interval than the conventional concrete, on average. Although contrary to the results of the RCPTs performed in the laboratory, these differences are consistent with the higher electrical conductivity values previously reported for the internally cured concrete. On average, at 5 months, 8 months, 1 year, and 2 years following deck construction, the conventional concrete bridge decks had 4.8, 6.6, 2.5 and 1.3 times more cracking, respectively, than the internally cured concrete decks. During the 1-year and 2-year distress surveys, very distinctive reflection cracks from the joints between the underlying pre-cast half-deck panels were observed on all of the decks. While the use of internally cured concrete did not prevent the occurrence of reflection cracking, it did apparently delay the propagation of such cracking.

5.3 Recommendations

Several recommendations can be derived from the results of this research. The use of pre-wetted LWFA to promote internal curing within concrete is recommended for reducing the occurrence of cracking in concrete bridge decks in Utah. However, as demonstrated in this research, internally cured concrete will not achieve its maximum potential in terms of crack reduction when half-deck concrete panels are used in deck construction. The use of internally cured concrete in monolithic concrete decks with conventional formwork may yield significantly better performance in this respect; studying additional decks involving different structural configurations, service conditions, and contractors and suppliers may also be of interest. Comparing the loss in deck service life from premature cracking with the benefits of accelerated construction resulting from the use of half-deck concrete panels is recommended for future deck designs. Further research evaluating chloride ingress in internally cured concrete decks should also be considered. Due to the elevated amount of cracking and high chloride concentration levels on the decks evaluated in this research, Utah DOT engineers may wish to apply a surface treatment to each of the bridge decks as soon as possible upon completion of this research.

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APPENDIX A DECK SENSOR LAYOUT MAPS

Appendix A contains layouts of the embedded sensors for each bridge deck. Each sensor location is labeled as northwest (NW), northeast (NE), southwest (SW), or southeast (SE).

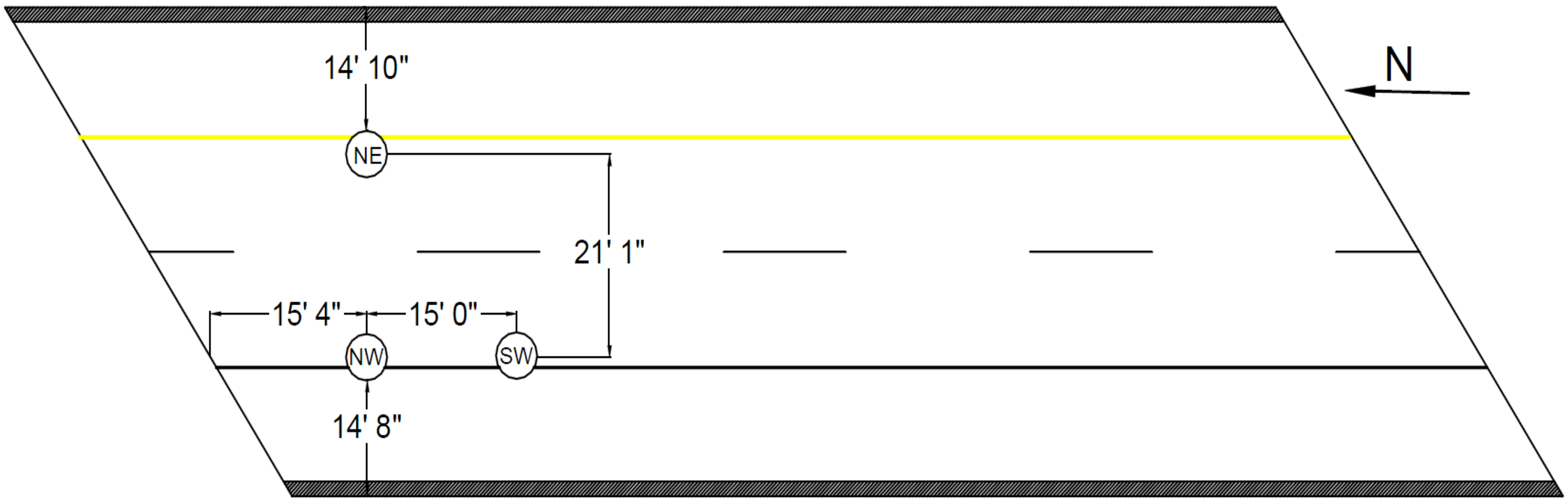

Figure A-1 Sensor locations on Dannon Way southbound conventional concrete deck.

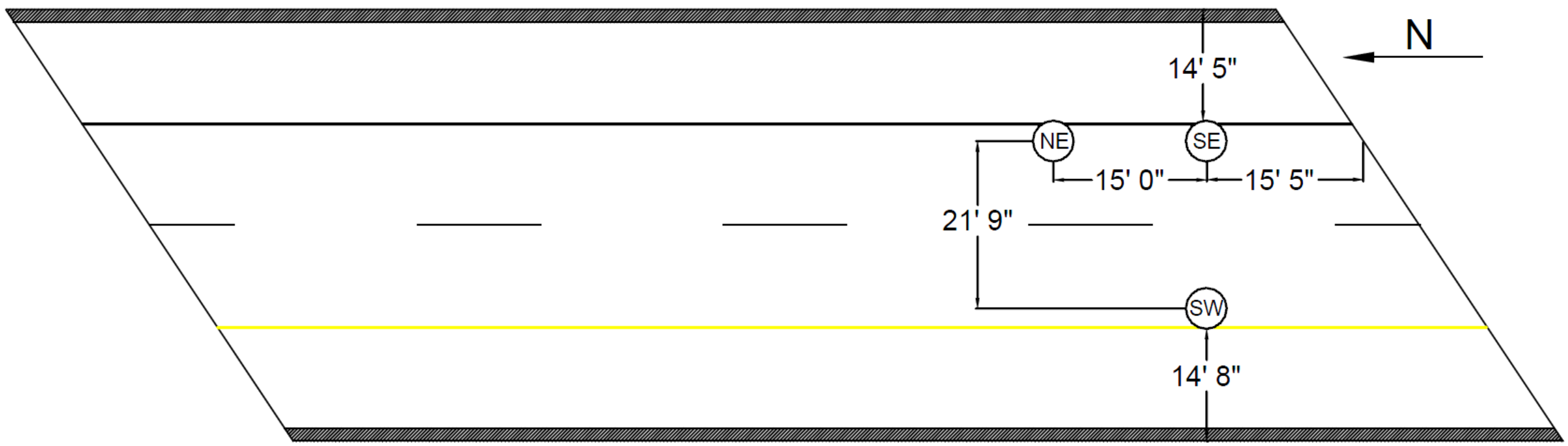

Figure A-2 Sensor locations on Dannon Way northbound internally cured concrete deck.

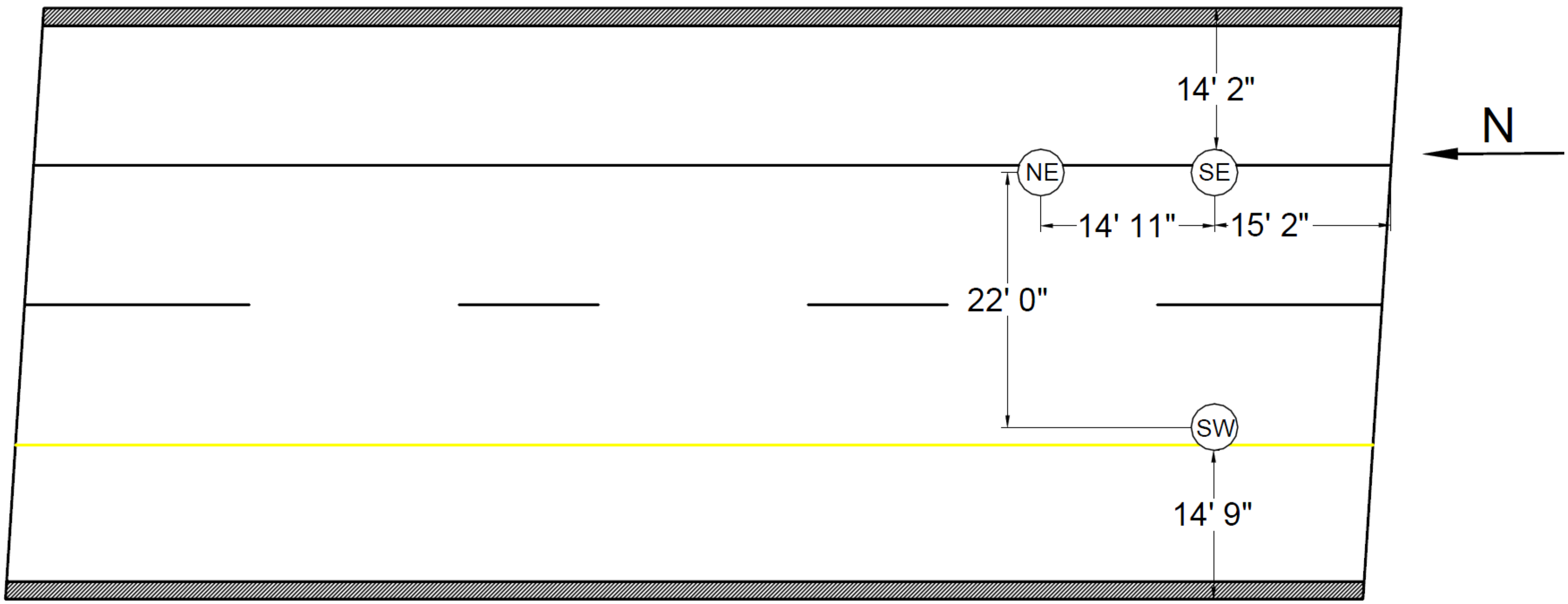

Figure A-3 Sensor locations on 8200 South northbound conventional concrete deck.

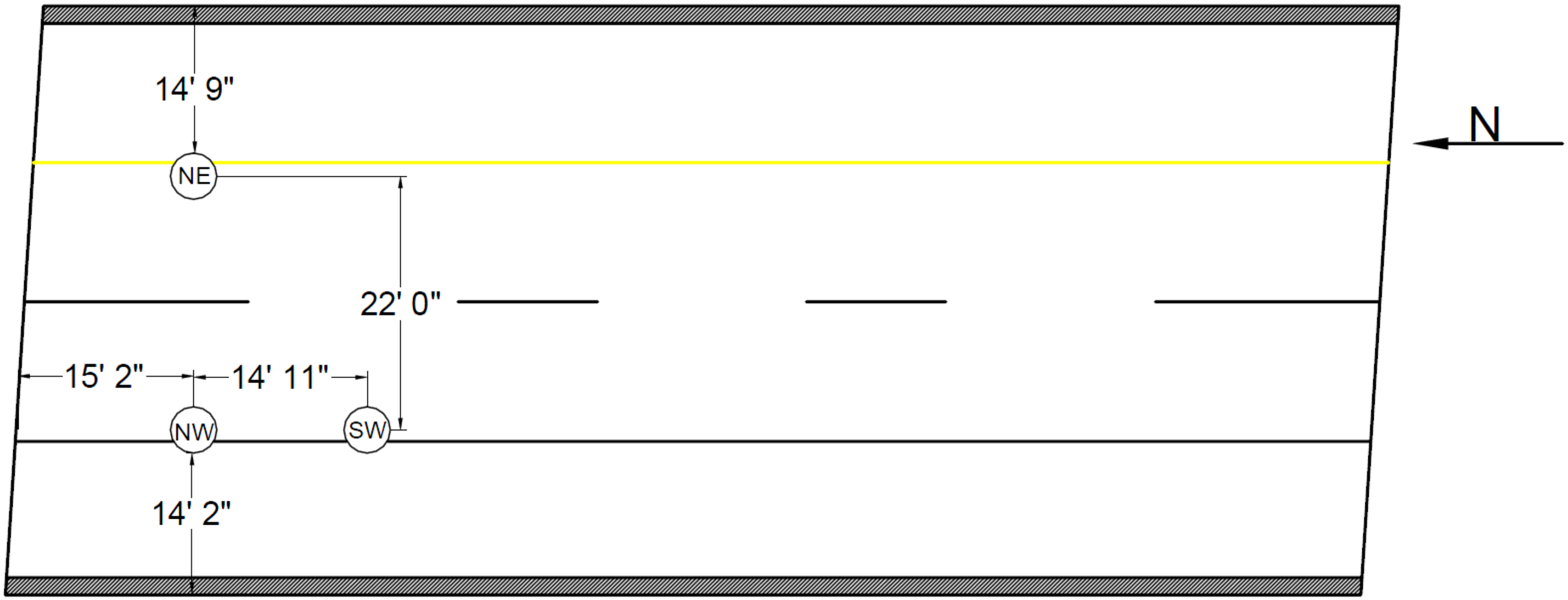

Figure A-4 Sensor locations on 8200 South southbound internally cured concrete deck.

APPENDIX B SENSOR, FIELD, AND LABORATORY DATA

Appendix B contains raw sensor, field, and laboratory data. Sensor locations are labeled as northwest (NW), northeast (NE), southwest (SW), or southeast (SE). (Hyphens in a table indicate that the given data were not measured or are not applicable.)

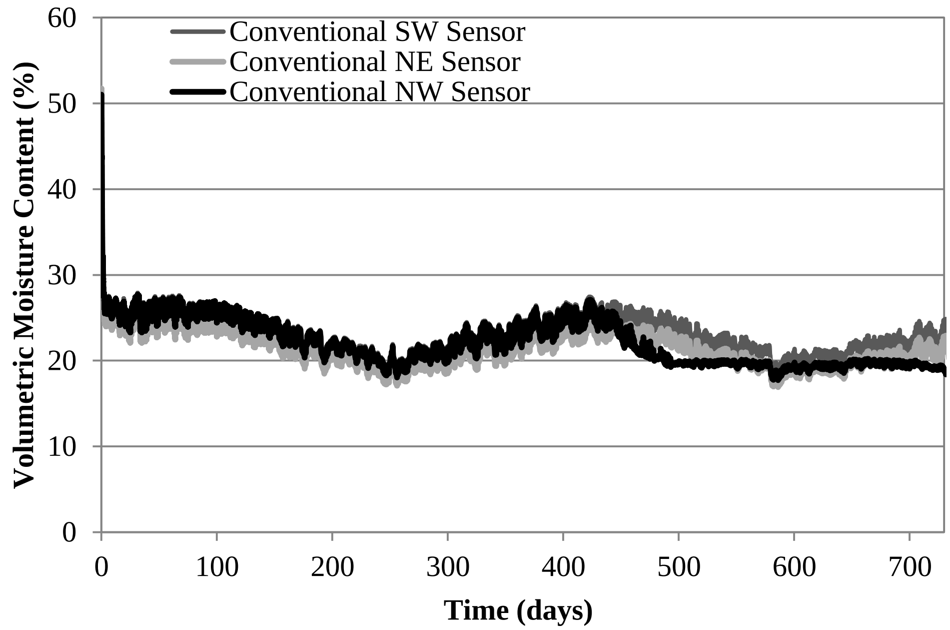

Figure B-1 Volumetric moisture content sensor readings at Dannon Way southbound conventional deck.

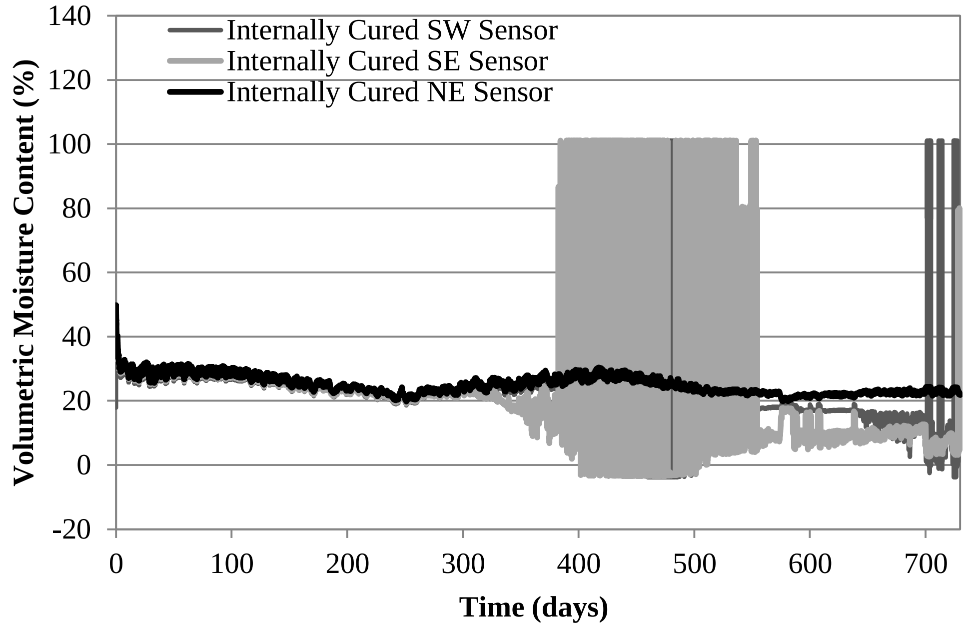

Figure B-2 Volumetric moisture content sensor readings at Dannon Way northbound internally cured deck.

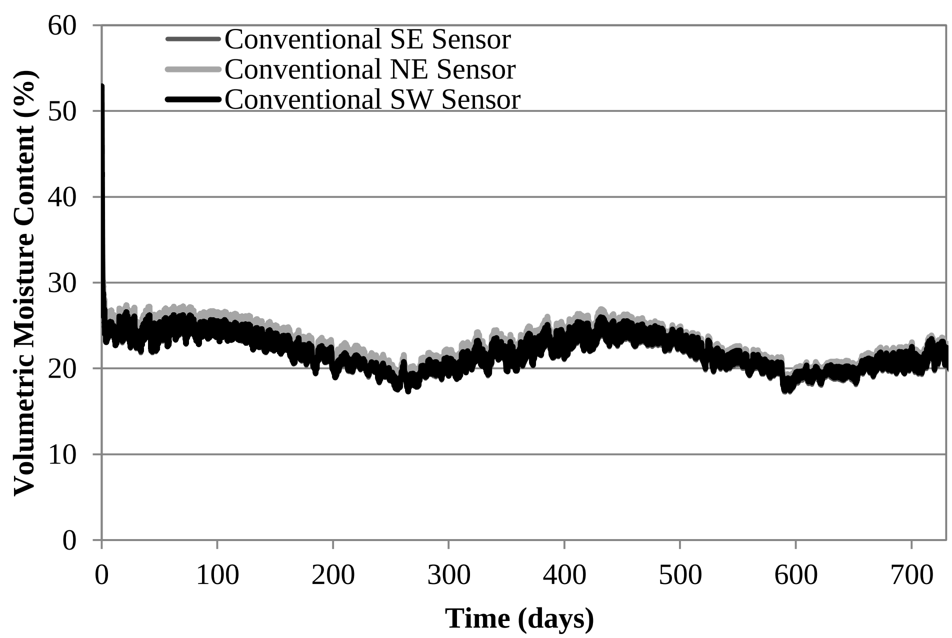

Figure B-3 Volumetric moisture content sensor readings at 8200 S northbound conventional concrete deck.

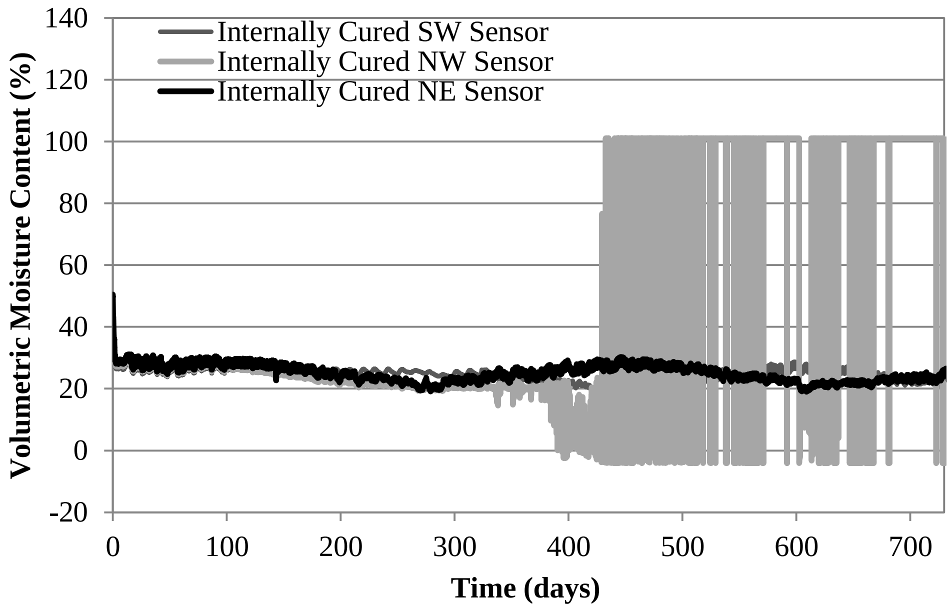

Figure B-4 Volumetric moisture content sensor readings at 8200 S southbound internally cured deck.

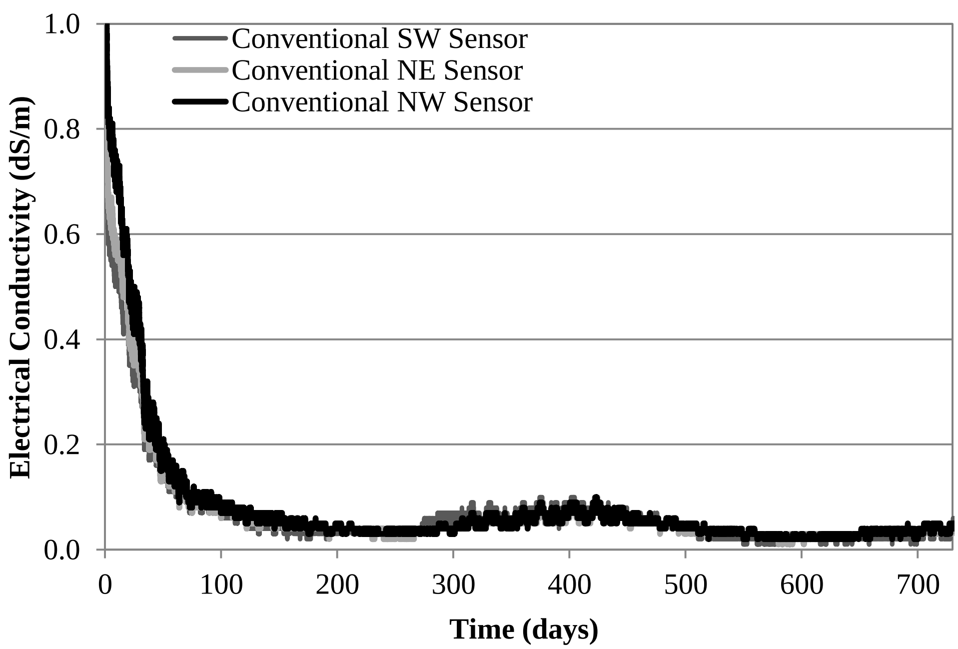

Figure B-5 Electrical conductivity sensor readings at Dannon Way southbound conventional deck.

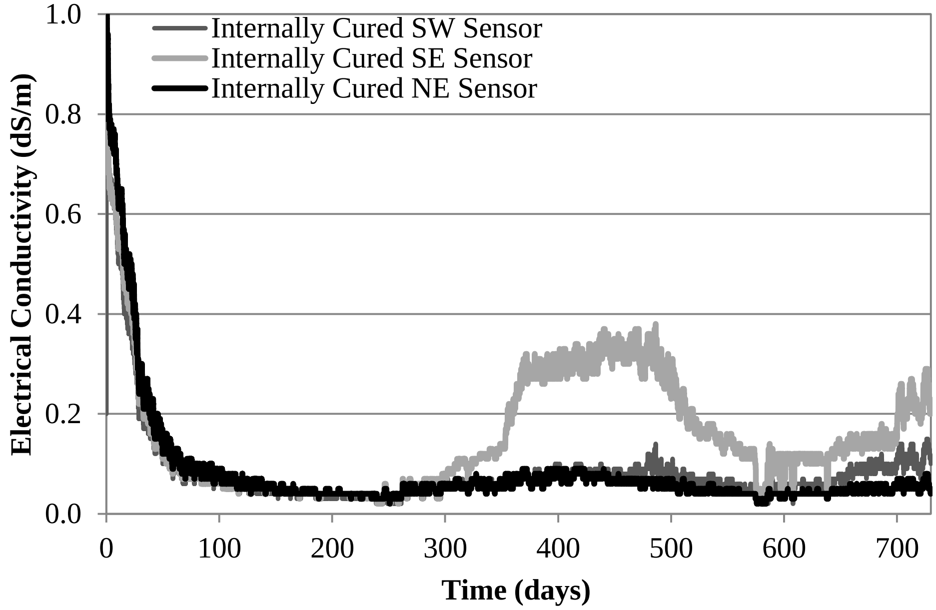

Figure B-6 Electrical conductivity sensor readings at Dannon Way northbound internally cured deck.

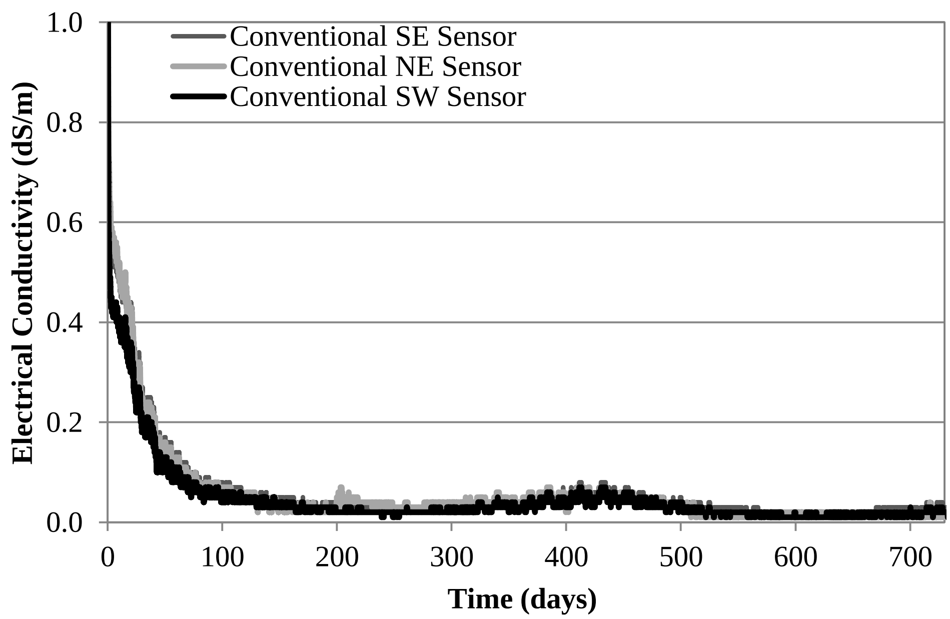

Figure B-7 Electrical conductivity sensor readings at 8200 S northbound conventional deck.

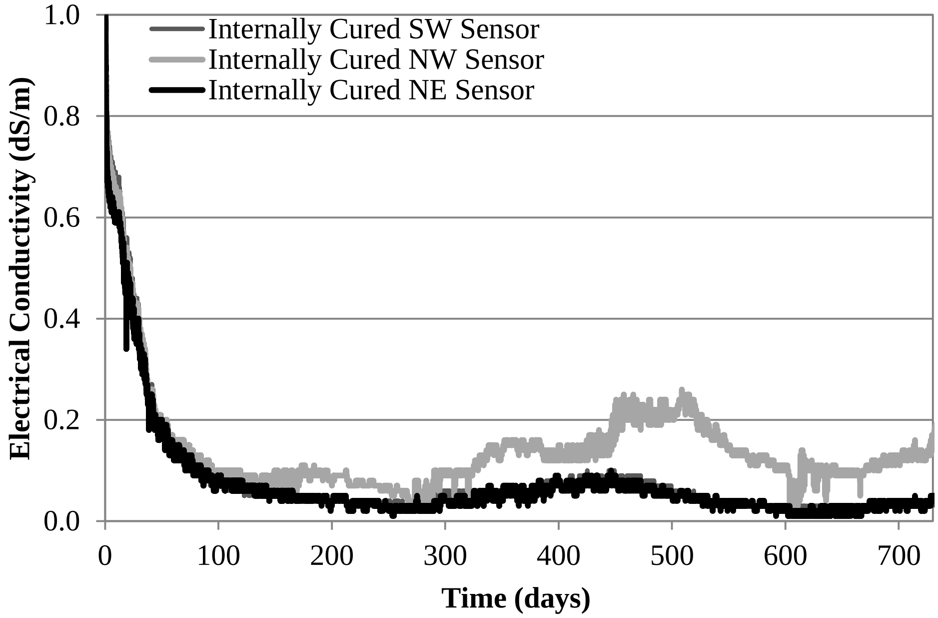

Figure B-8 Electrical conductivity sensor readings at 8200 S southbound internally cured deck.

Table B-1 Slump and Air Content Measurements

Bridge Location	Slump* (in.)		Air Content* (%)	
	Avg.	St. Dev.	Avg.	St. Dev.
Dannon Way SB Conventional	4.0	0.34	6.0	0.14
Dannon Way NB Internally Cured	3.4	0.42	6.4	0.79
8200 South NB Conventional	3.9	0.26	5.7	0.14
8200 South SB Internally Cured	3.2	0.21	6.0	0.21

* Data obtained from 20 measurements per deck for the Dannon Way decks and 16 measurements per deck for the 8200 South decks

Table B-2 Cover Depth Measurements

Bridge Location	Sensor Location	Cover Depth (in.)				
		North Transverse	South Transverse	East Longitudinal	West Longitudinal	Avg.
Dannon Way SB Conventional	NW	3.40	3.35	3.45	3.35	3.39
	NE	3.45	3.45	3.40	3.35	3.41
	SW	2.50	3.45	3.50	3.50	3.24
Dannon Way NB Internally Cured	SE	2.75	2.75	2.80	2.85	2.79
	NE	2.70	2.65	2.70	2.80	2.71
	SW	2.85	2.75	2.80	2.85	2.81
8200 South NB Conventional	SE	2.65	2.70	2.90	2.90	2.79
	NE	2.75	2.70	2.80	2.80	2.76
	SW	2.45	2.45	2.70	2.65	2.56
8200 South SB Internally Cured	NW	2.25	2.30	2.30	2.35	2.30
	NE	2.35	2.40	2.30	2.50	2.39
	SW	2.45	2.45	2.50	2.60	2.50

Table B-3 Example Data Logger Output

5G0C1283-390654348	Port 1	Port 1	Port 1	Port 2	Port 2	Port 2	Port 3	Port 3	Port 3	Port 4	Port 4
	5TE Moisture Temp EC	5TE Moisture Temp EC	5TE Moisture Temp EC	5TE Moisture Temp EC	5TE Moisture Temp EC	5TE Moisture Temp EC	5TE Moisture Temp EC	5TE Moisture Temp EC	5TE Moisture Temp EC	EHT RH/ Temp	EHT RH/ Temp
Measurement Time	% VWC	Temp °F	EC dS/m	% VWC	Temp °F	EC dS/m	% VWC	Temp °F	EC dS/m	Temp °F	Aw
11/10/2012 13:00	21.73	34.16	0.03	21.18	35.78	0.08	22.71	35.42	0.03	44.42	0.54
11/10/2012 14:00	21.88	35.96	0.03	21.29	38.12	0.08	22.88	37.4	0.03	42.26	0.55
11/10/2012 15:00	21.99	36.86	0.04	21.37	39.02	0.08	22.95	38.3	0.03	40.10	0.58
11/10/2012 16:00	21.99	37.04	0.03	21.33	39.02	0.08	22.95	38.12	0.03	38.66	0.58
11/10/2012 17:00	21.95	36.14	0.03	21.26	37.94	0.08	22.92	37.22	0.03	33.98	0.66
11/10/2012 18:00	21.84	34.88	0.03	21.18	36.50	0.08	22.78	35.96	0.03	30.92	0.73
11/10/2012 19:00	21.73	33.80	0.03	21.11	35.06	0.08	22.67	34.88	0.03	28.58	0.71
11/10/2012 20:00	21.62	32.72	0.03	21.03	33.44	0.08	22.53	33.44	0.03	26.96	0.70
11/10/2012 21:00	21.55	31.82	0.03	21.00	32.36	0.08	22.42	32.18	0.03	30.38	0.76
11/10/2012 22:00	21.51	31.64	0.03	20.96	32.00	0.08	22.38	32.00	0.02	28.94	0.80

Table B-4 Concrete Compressive Strength Measurements

Bridge Location	Sensor Location	Compressive Strength (psi)						
		7-day	28-day	58-day*	3-month	6-month	1-year	1-year (Fog Room Only)
Dannon Way SB Conventional	NW	4239	5191	6194	6851	6056	7449	6652
	NE	3967	5456	6088	4191	6915	6566	5375
	SW	4507	6009	6930	6689	5539	7350	6265
Dannon Way NB Internally Cured	SE	4098	6314	6752	6780	6315	6572	5442
	NE	4316	5169	6150	7031	-	6508	5575
	SW	4160	5484	5780	5367	-	5719	6402
8200 South NB Conventional	SE	4625	4521	5187	6554	6476	7172	6581
	NE	4383	5143	6574	6242	7413	7707	7282
	SW	4121	6103	5026	6618	6574	6235	6995
8200 South SB Internally Cured	NW	3976	4668	4968	5880	-	5161	6188
	NE	3901	5528	5575	6092	-	6579	6087
	SW	4232	5559	6051	6378	6481	6467	6744

* Compressive strength testing following 30 days of freeze-thaw cycling

Table B-5 Comparison of Concrete Compressive Strength Measurements

Location	Compressive Strength (psi)							
	7-day				28-day			
	BYU		Utah DOT		BYU		Utah DOT	
	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.
Dannon Way SB Conventional	4238	270	4350	153	5552	417	5495	235
Dannon Way NB Internally Cured	4191	112	3778	314	5656	592	5370	364
8200 South NB Conventional	4377	112	4398	725	5256	797	5823	378
8200 South SB Internally Cured	4036	173	4185	266	5252	506	5708	719

Table B-6 Rapid Chloride Permeability Measurements

Bridge Location	Sensor Location	Rapid Chloride Permeability (coulombs)			
		28-day	6-month	1-year	1-year (Fog Room Only)
Dannon Way SB Conventional	NW	1281	1285	1017	377
	NE	2034	1485	1253	397
	SW	1431	1353	1566	418
Dannon Way NB Internally Cured	SE	1113	823	819	327
	NE	1323	1173	1148	385
	SW	1260	1221	941	376
8200 South NB Conventional	SE	1304	1105	1584	381
	NE	1318	1050	823	340
	SW	1134	1146	1270	342
8200 South SB Internally Cured	NW	1145	1294	1403	401
	NE	1112	1121	987	385
	SW	1057	820	1059	352

Table B-7 Properties of Specimens Cured in Open Air for Free-Free Resonant Testing at 1 Year

Bridge Location	Sensor Location	Length (in.)				Diameter (in.)				Weight (lb)	Density (lb/ft^3)
		1	2	3	Avg.	1	2	3	Avg.		
Dannon Way SB Conventional	NW	8.1490	8.1215	8.1715	8.1473	4.0410	4.0325	4.0045	4.0260	8.2125	136.8
	NE	8.0905	8.1265	8.0940	8.1037	4.0270	4.0170	4.0025	4.0155	8.1445	137.1
	SW	8.1512	8.1730	8.1925	8.1722	4.0370	4.0120	3.9925	4.0138	8.0645	134.8
Dannon Way NB Internally Cured	SE	8.0830	8.0500	8.1000	8.0777	4.0030	3.9970	3.9920	3.9973	7.9045	134.7
	NE	8.0520	8.0975	8.0455	8.0650	4.0275	4.0085	3.9910	4.0090	7.7575	131.7
	SW	8.0785	8.0620	8.1500	8.0968	4.0065	3.9935	3.9795	3.9932	7.7025	131.3
8200 South NB Conventional	SE	8.0595	8.0730	8.0540	8.0622	3.9945	4.0090	4.0200	4.0078	8.0685	137.1
	NE	8.0400	8.0545	8.0485	8.0477	4.0205	4.0085	3.9915	4.0068	8.1225	138.3
	SW	8.0240	8.0270	8.0710	8.0407	4.0150	4.0000	3.9960	4.0037	8.1385	138.9
8200 South SB Internally Cured	NW	8.1200	8.1095	8.1320	8.1205	4.0250	4.0135	3.9925	4.0103	7.5585	127.3
	NE	8.0170	8.0315	7.9940	8.0142	3.9920	4.0095	4.0230	4.0082	7.5705	129.4
	SW	8.0545	8.0880	8.0605	8.0677	4.0000	4.0085	4.0195	4.0093	7.6680	130.1

Table B-8 Frequency and Modulus Values of Specimens Cured in Open Air for Free-Free Resonant Testing at 1 Year

Bridge Location	Sensor Location	Resonant Frequency (Hz)							Modulus (psi)
		1	2	3	4	5	6	Avg.	
Dannon Way SB Conventional	NW	9535	9535	9535	9535	9535	9535	9535	4946621
	NE	9766	9766	9766	9766	9766	9766	9766	5145145
	SW	9537	9537	9537	9537	9537	9537	9537	4904475
Dannon Way NB Internally Cured	SE	9887	9867	9867	9867	9867	9867	9870	5130752
	NE	9751	9751	9751	9694	9694	9694	9722	4849864
	SW	9694	9694	9694	9694	9694	9694	9694	4844024
8200 South NB Conventional	SE	9458	9458	9458	9458	9497	9497	9471	4787785
	NE	9751	9751	9694	9694	9694	9751	9722	5072624
	SW	9809	9751	9809	9751	9751	9751	9770	5136612
8200 South SB Internally Cured	NW	9405	9405	9347	9405	9347	9405	9386	4431288
	NE	9751	9751	9694	9694	9694	9751	9722	4705079
	SW	9809	9751	9809	9751	9751	9751	9770	4842191

Table B-9 Properties of Specimens Cured in Fog Room Only for Free-Free Resonant Testing at 1 Year

Bridge Location	Sensor Location	Length (in.)				Diameter (in.)				Weight (lb)	Density (lb/ft^3)
		1	2	3	Avg.	1	2	3	Avg.		
Dannon Way SB Conventional	NW	8.160	8.165	8.084	8.136	4.044	4.009	4.001	4.018	8.475	142.0
	NE	8.141	8.137	8.223	8.167	4.034	4.019	4.009	4.020	8.397	140.0
	SW	8.134	8.129	8.133	8.132	3.979	4.004	4.002	3.995	8.527	144.6
Dannon Way NB Internally Cured	SE	8.039	8.067	8.050	8.052	4.026	4.010	3.998	4.011	8.269	140.4
	NE	8.026	8.079	8.100	8.068	3.989	4.004	4.020	4.004	8.181	139.2
	SW	8.098	8.094	8.135	8.109	4.022	4.048	4.070	4.046	8.194	135.8
8200 South NB Conventional	SE	8.045	7.979	8.023	8.015	4.058	4.034	4.010	4.034	8.329	140.5
	NE	8.103	8.114	8.150	8.122	3.998	4.024	4.055	4.026	8.533	142.6
	SW	8.051	8.045	8.036	8.044	4.049	4.016	3.998	4.021	8.416	142.4
8200 South SB Internally Cured	NW	8.085	8.122	8.108	8.105	4.000	4.008	4.032	4.013	7.937	133.8
	NE	8.121	8.091	8.051	8.087	4.004	4.015	4.024	4.014	7.917	133.7
	SW	8.036	7.949	8.054	8.013	3.994	4.004	4.036	4.011	7.982	136.2

Table B-10 Frequency and Modulus Values of Specimens Cured in Fog Room Only for Free-Free Resonant Testing at 1 Year

Bridge Location	Sensor Location	Resonant Frequency (Hz)							Modulus (psi)
		1	2	3	4	5	6	Avg.	
Dannon Way SB Conventional	NW	9151	9151	9189	9151	9151	9151	9157	4720890
	NE	9151	9112	9151	9151	9151	9151	9144	4676308
	SW	9035	8997	8997	8997	8997	8997	9003	4642381
Dannon Way NB Internally Cured	SE	10328	10328	10328	10328	10328	10328	10328	5818211
	NE	10213	10213	10213	10213	10213	10213	10213	5660233
	SW	10097	10097	10097	10097	10097	10097	10097	5453265
8200 South NB Conventional	SE	9151	9151	9189	9151	9151	9151	9157	4533953
	NE	9151	9112	9151	9151	9151	9151	9144	4713541
	SW	9035	8997	8997	8997	8997	8997	9003	4473392
8200 South SB Internally Cured	NW	9867	9867	9867	9867	9867	9867	9867	5125106
	NE	9867	9924	9867	9867	9867	9867	9876	5108449
	SW	9867	9867	9867	9867	9867	9867	9867	5100214

Table B-11 Moisture Content Measurements at 1 Year

Bridge Location	Sensor Location	Water Content (%)
Dannon Way SB Open Air Conventional	NW	2.45
	NE	2.35
	SW	2.46
Dannon Way SB Fog Room Only Conventional	NW	6.21
	NE	6.60
	SW	5.93
Dannon Way NB Open Air Internally Cured	SE	3.34
	NE	3.05
	SW	2.91
Dannon Way NB Fog Room Only Internally Cured	SE	8.12
	NE	8.62
	SW	8.35
8200 South NB Open Air Conventional	SE	2.67
	NE	3.20
	SW	2.77
8200 South NB Fog Room Only Conventional	SE	6.93
	NE	6.47
	SW	6.36
8200 South SB Open Air Internally Cured	NW	3.34
	NE	3.33
	SW	3.50
8200 South SB Fog Room Only Internally Cured	NW	8.98
	NE	9.03
	SW	8.71

Table B-12 Schmidt Rebound Numbers on Dannon Way Southbound Conventional Concrete Deck at 1 Year

Location Number*	Test Location	Schmidt Rebound Number			
		1	2	3	Avg.
1	25' from N	47	44	41	44.0
2	75' from N	48	48	44	46.7
3	25' from S	46	46	47	46.3
4	75' from S	48	58	54	53.3
-	NW Sensor	51	47	46	48.0
-	SW Sensor	52	50	48	50.0
-	NE Sensor	47	51	55	51.0

***** As shown in Figure 3-17

Table B-13 Schmidt Rebound Numbers on Dannon Way Northbound Internally Cured Concrete Deck at 1 Year

Location Number*	Test Location	Schmidt Rebound Number			
		1	2	3	Avg.
1	25' from N	45	46	40	43.7
2	75' from N	42	42	45	43.0
3	25' from S	52	52	49	51.0
4	75' from S	40	39	41	40.0
-	SE Sensor	50	62	45	52.3
-	NE Sensor	46	47	52	48.3
-	SW Sensor	46	46	48	46.7

***** As shown in Figure 3-17

Table B-14 Schmidt Rebound Numbers on 8200 South Northbound Conventional Concrete Deck at 1 Year

Location Number*	Test Location	Schmidt Rebound Number			
		1	2	3	Avg.
1	25' from N	49	48	51	49.3
2	75' from N	49	47	46	47.3
3	25' from S	-	-	-	-
4	75' from S	44	41	46	43.7
-	SE Sensor	47	46	48	47.0
-	NE Sensor	48	45	50	47.7
-	SW Sensor	46	51	47	48.0

***** As shown in Figure 3-18

Table B-15 Schmidt Rebound Numbers on 8200 South Southbound Internally Cured Concrete Deck at 1 Year

Location Number*	Test Location	Schmidt Rebound Number			
		1	2	3	Avg.
1	25' from N	44	50	44	46.0
2	75' from N	46	46	44	45.3
3	25' from S	43	42	45	43.3
4	75' from S	54	44	42	46.7
-	NW Sensor	50	49	52	50.3
-	SW Sensor	52	47	46	48.3
-	NE Sensor	50	60	56	55.3

* As shown in Figure 3-18

Table B-16 Compressive Strength Estimations from Schmidt Rebound Hammer Testing at 1 Year

Bridge Location	Compressive Strength (psi)							
	1	2	3	4	5	6	7	Avg.
Dannon Way SB Conventional	6450	6800	6800	7700	7250	7700	7700	7200
Dannon Way NB Internally Cured	7500	7000	-	6200	7000	7000	7000	6950
8200 South NB Conventional	6200	6200	7700	5500	7700	7250	6800	6764
8200 South SB Internally Cured	6800	6600	6200	6800	7700	7250	7700	7007

Table B-17 Schmidt Rebound Numbers on Dannon Way Southbound Conventional Concrete Deck at 2 Years

Location Number*	Test Location	Schmidt Rebound Number			
		1	2	3	Avg.
1	25' from N	48	49	52	49.7
2	75' from N	53	50	51	51.3
3	25' from S	45	48	46	46.3
4	75' from S	53	55	58	55.3
-	NW Sensor	54	57	53	54.7
-	SW Sensor	57	60	57	58.0
-	NE Sensor	52	50	48	50.0

***** As shown in Figure 3-17

Table B-18 Schmidt Rebound Numbers on Dannon Way Northbound Internally Cured Concrete Deck at 2 Years

Location Number*	Test Location	Schmidt Rebound Number			
		1	2	3	Avg.
1	25' from N	59	51	58	56.0
2	75' from N	42	51	48	47.0
3	25' from S	48	46	45	46.3
4	75' from S	46	52	54	50.7
-	SE Sensor	52	57	51	53.3
-	NE Sensor	44	46	46	45.3
-	SW Sensor	46	46	50	47.3

***** As shown in Figure 3-17

Table B-19 Schmidt Rebound Numbers on 8200 South Northbound Conventional Concrete Deck at 2 Years

Location Number*	Test Location	Schmidt Rebound Number			
		1	2	3	Avg.
1	25' from N	61	50	56	55.7
2	75' from N	54	53	54	53.7
3	25' from S	53	50	53	52.0
4	75' from S	51	54	47	50.7
-	SE Sensor	52	53	52	52.3
-	NE Sensor	45	48	48	47.0
-	SW Sensor	50	52	51	51.0

***** As shown in Figure 3-18

Table B-20 Schmidt Rebound Numbers on 8200 South Southbound Internally Cured Concrete Deck at 2 Years

Location Number*	Test Location	Schmidt Rebound Number			
		1	2	3	Avg.
1	25' from N	54	47	54	51.7
2	75' from N	54	55	54	54.3
3	25' from S	52	50	52	51.3
4	75' from S	45	46	47	46.0
-	NW Sensor	47	51	46	48.0
-	SW Sensor	56	58	56	56.7
-	NE Sensor	50	54	51	51.7

***** As shown in Figure 3-18

Table B-21 Compressive Strength Estimations from Schmidt Rebound Hammer Testing at 2 Years

Bridge Location	Compressive Strength (psi)							
	1	2	3	4	5	6	7	Avg.
Dannon Way SB Conventional	7500	7700	6800	7700	7700	7700	7700	7543
Dannon Way NB Internally Cured	7700	7000	6800	7700	7700	6600	7000	7214
8200 South NB Conventional	7700	7700	7700	7700	7700	7000	7700	7600
8200 South SB Internally Cured	7700	7700	7700	6800	7250	7700	7700	7507

Table B-22 Chloride Concentration Measurements at 1 Year

Bridge Location	Depth (in.)	Chloride Concentration (lb Cl^-/yd^3 Concrete)				
		1	2	3	4	Avg.
Dannon Way SB Conventional	0.0 to 0.5	5.25	6.39	11.4	10.3	8.34
	0.5 to 1.0	1.01	3.18	2.8	0.66	1.91
Dannon Way NB Internally Cured	0.0 to 0.5	7.64	8.55	8.55	9.66	8.6
	0.5 to 1.0	1.06	3.61	0.88	2.36	1.98
8200 South NB Conventional	0.0 to 0.5	6.65	6.44	10.71	10.48	8.57
	0.5 to 1.0	2.68	1.51	1.35	0.84	1.59
8200 South SB Internally Cured	0.0 to 0.5	5.79	6.46	10.13	13.81	9.05
	0.5 to 1.0	1.53	0.68	3.84	2.73	2.2

Table B-23 Chloride Concentration Measurements at 2 Years

Bridge Location	Depth (in)	Chloride Concentration (lb Cl^-/yd^3 Concrete)				
		1	2	3	4	Avg.
Dannon Way SB Conventional	0.0 to 0.5	11.26	12.52	10.22	8.89	10.72
	0.5 to 1.0	0.48	5.20	1.29	0.86	1.96
Dannon Way NB Internally Cured	0.0 to 0.5	10.46	10.64	11.42	12.28	11.20
	0.5 to 1.0	4.39	2.39	2.52	7.13	4.11
8200 South NB Conventional	0.0 to 0.5	9.22	8.45	11.59	13.85	10.78
	0.5 to 1.0	1.25	1.79	0.70	2.62	1.59
8200 South SB Internally Cured	0.0 to 0.5	12.41	15.94	13.55	11.41	13.33
	0.5 to 1.0	3.93	3.11	0.66	2.26	2.49

APPENDIX C DECK DISTRESS DATA

Appendix C provides pictorial evidence of typical crack lengths and widths and also provides full distress survey maps prepared for each deck upon completion of these inspections. The honeycomb pattern at selected locations on the distress maps represents map cracking. The thick solid line on each distress map positioned approximately 5 to 7 ft from the end of the deck and oriented parallel with the bridge skew represents the location of the metal shoring used in construction.

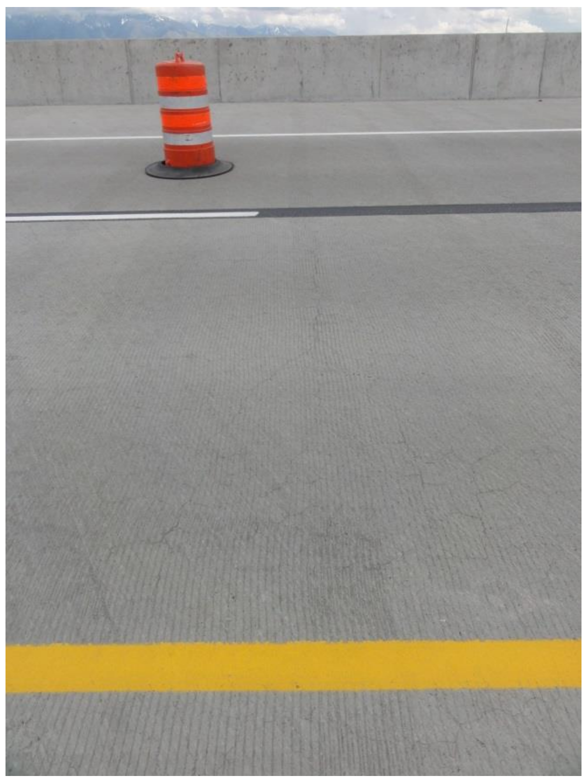

Figure C-1 Reflection cracking from half-deck concrete panels in Dannon Way northbound internally cured concrete deck at 1 year.

Figure C-2 Reflection cracking from half-deck concrete panels in Dannon Way northbound internally cured concrete deck at 2 years.

Figure C-3 Cracking in Dannon Way northbound internally cured concrete deck at 1 year.

Figure C-4 Cracking in Dannon Way northbound internally cured concrete deck at 2 years.

Figure C-5 Reflection cracking from half-deck concrete panels in 8200 South northbound conventional concrete deck at 1 year.

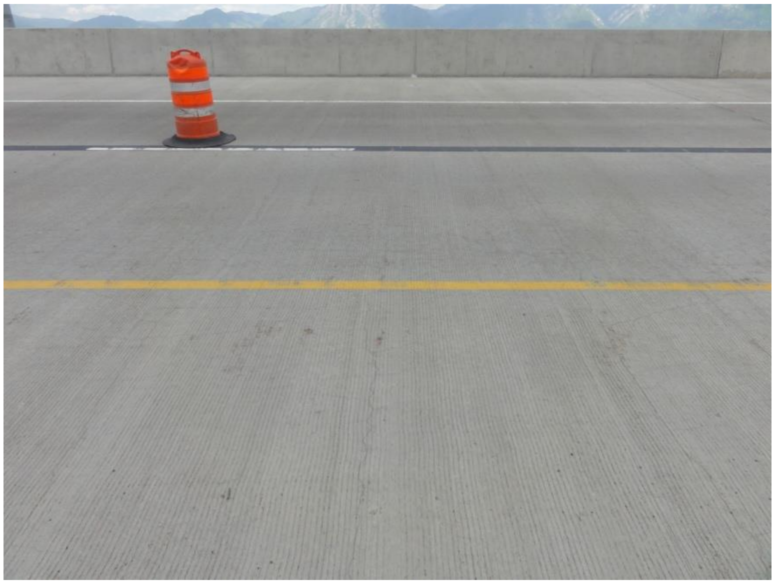

Figure C-6 Reflection cracking from half-deck concrete panels in 8200 South northbound conventional concrete deck at 2 years.

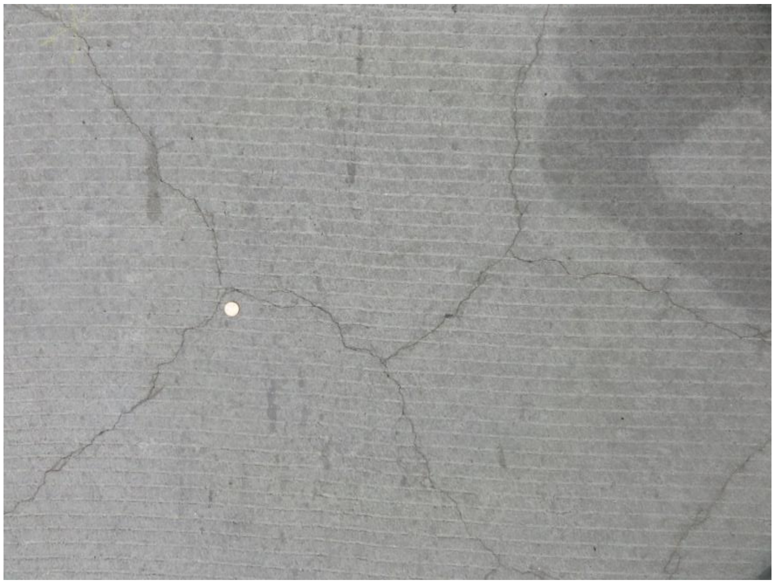

Figure C-7 Cracking in 8200 South northbound conventional concrete deck at 1 year.

Figure C-8 Cracking in 8200 South northbound conventional concrete deck at 2 years.

Figure C-9 Reflection and map cracking in 8200 South northbound conventional concrete deck at 1 year.

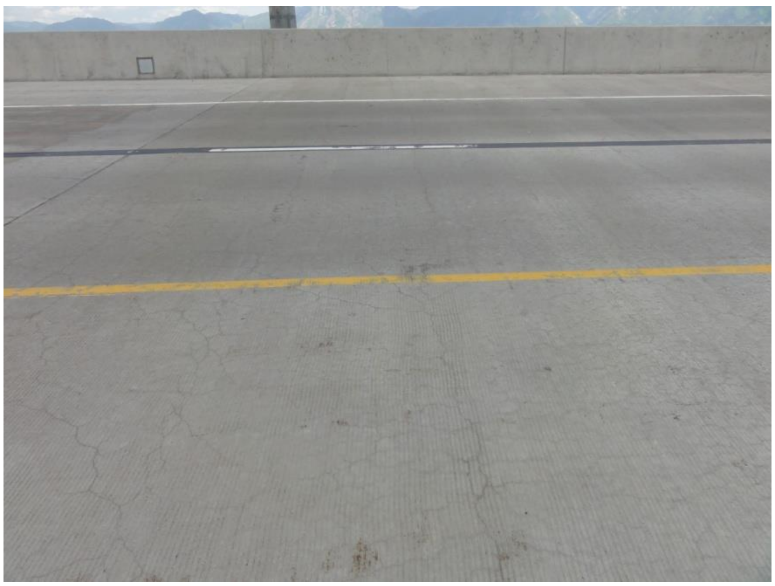

Figure C-10 Reflection and map cracking in 8200 South northbound conventional concrete deck at 2 years.

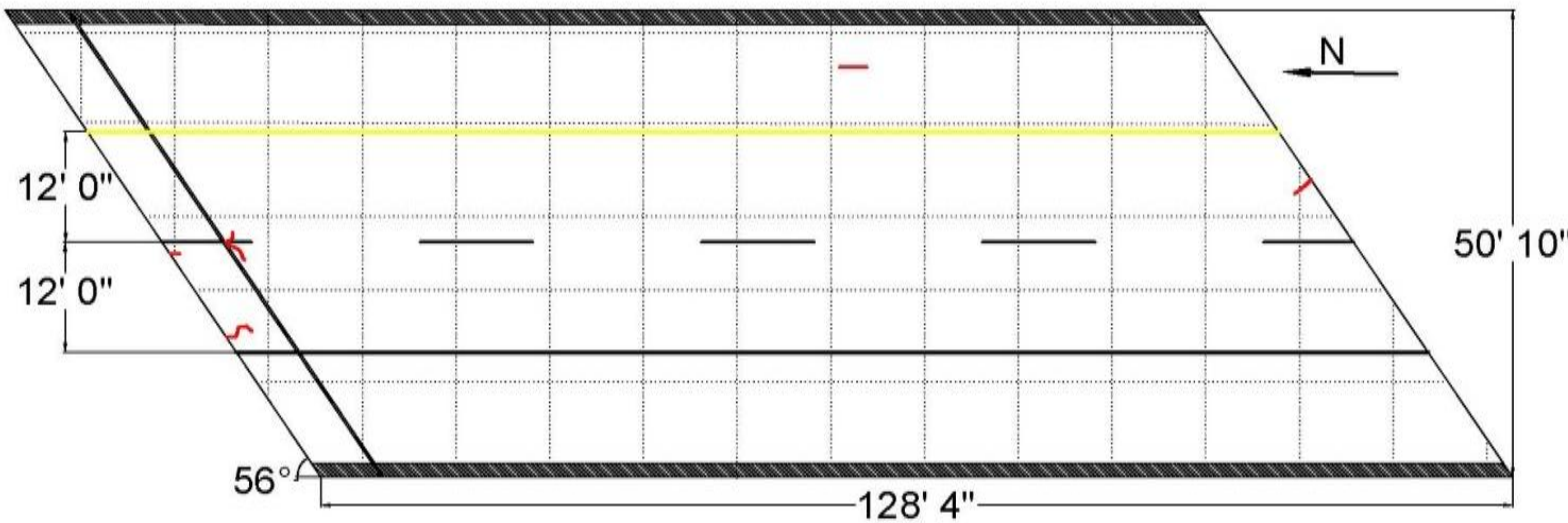

Figure C-11 Distress map for Dannon Way southbound conventional concrete deck at 5 months.

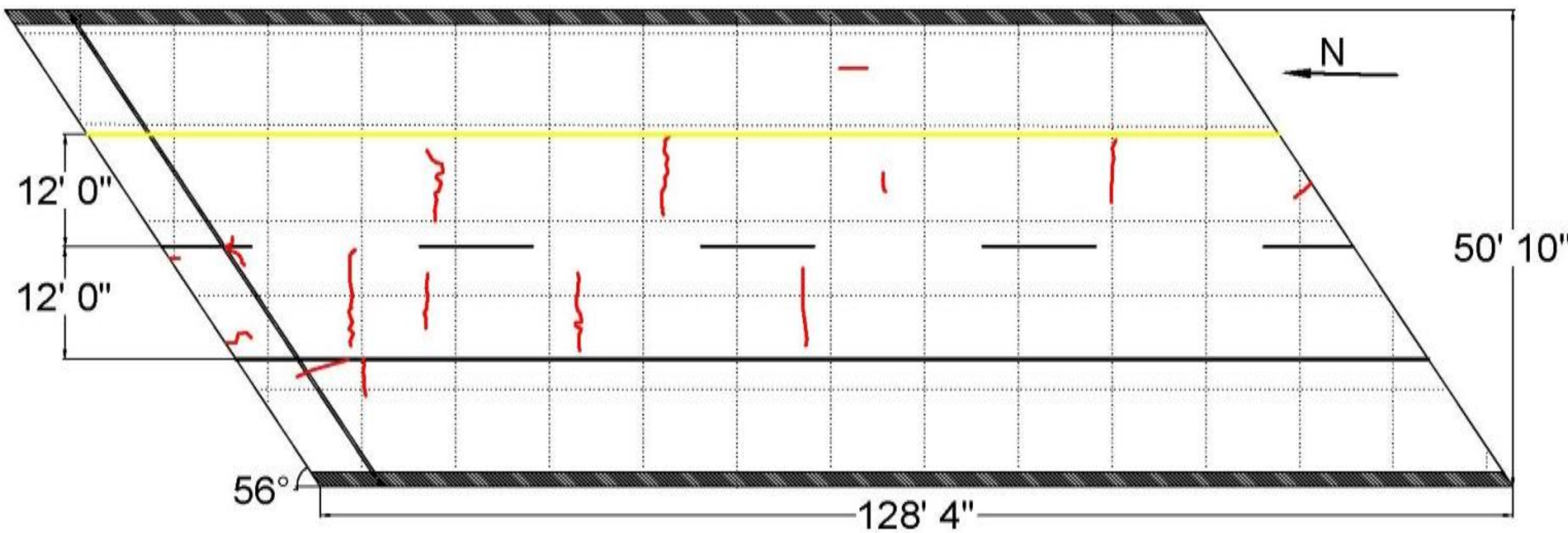

Figure C-12 Distress map for Dannon Way southbound conventional concrete deck at 8 months.

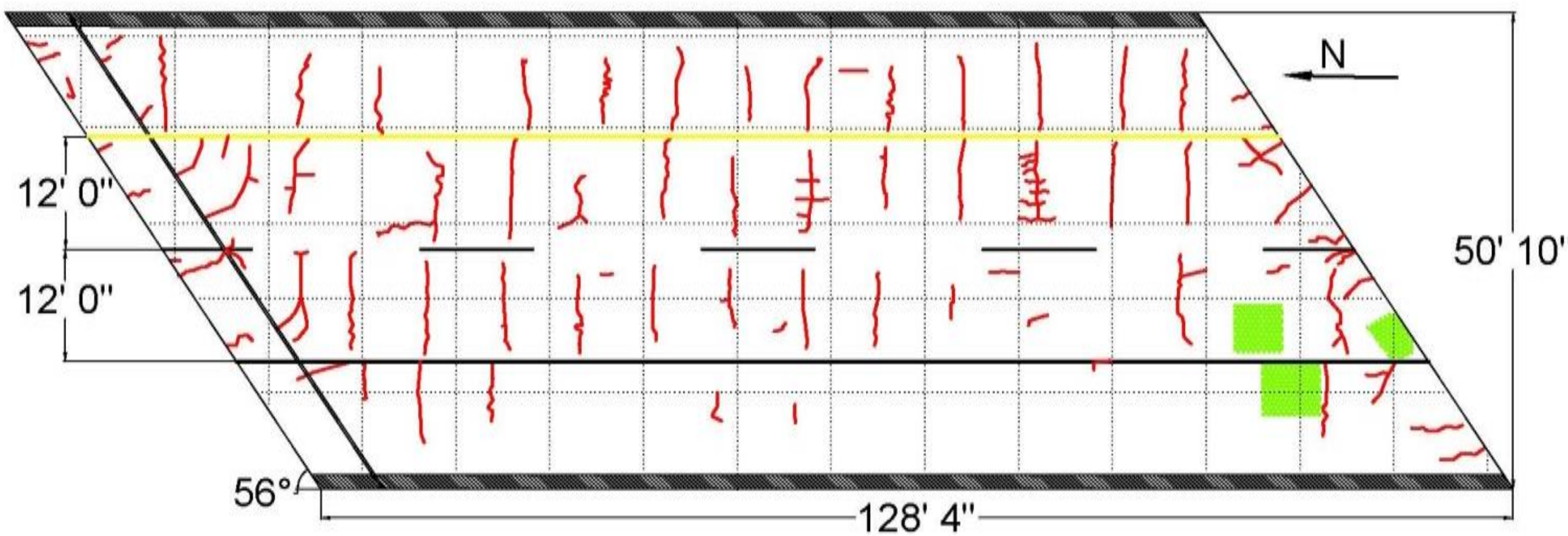

Figure C-13 Distress map for Dannon Way southbound conventional concrete deck at 1 year.

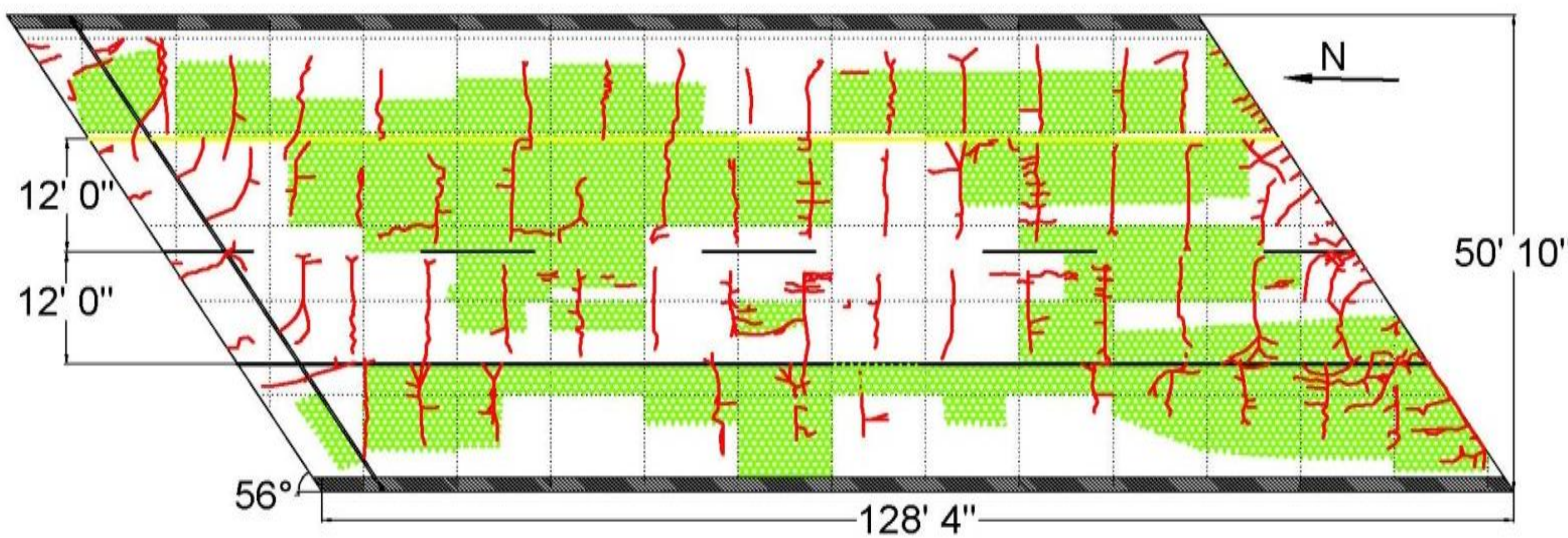

Figure C-14 Distress map for Dannon Way southbound conventional concrete deck at 2 years.

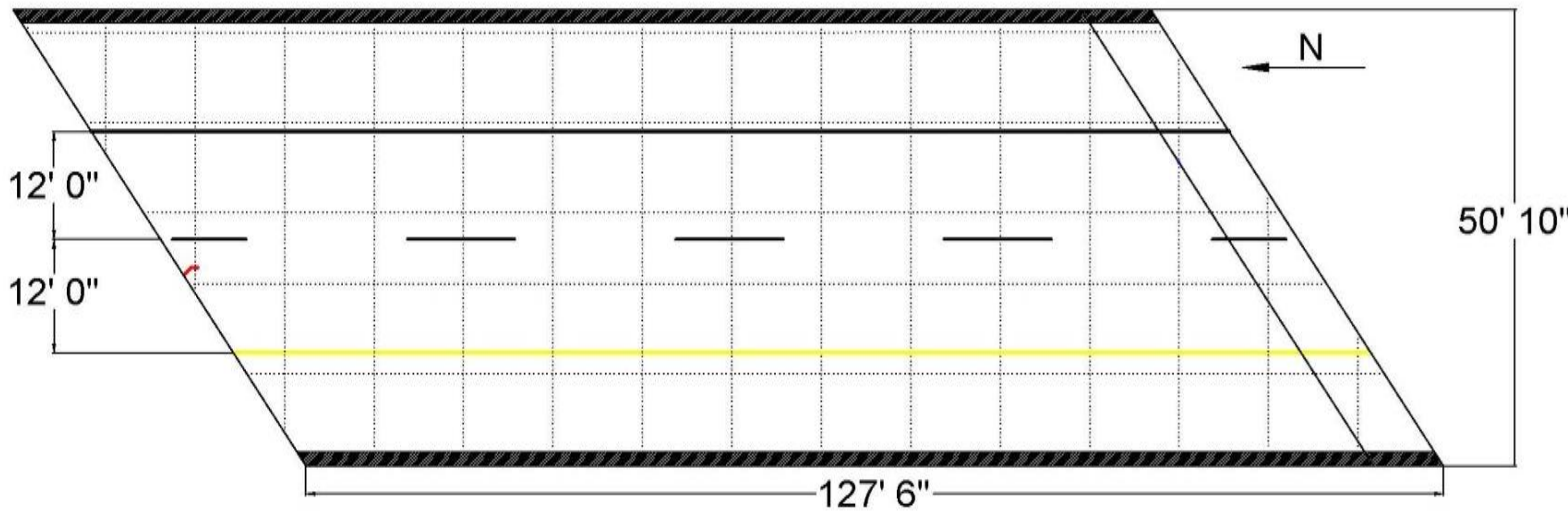

Figure C-15 Distress map for Dannon Way northbound internally cured concrete deck at 5 months.

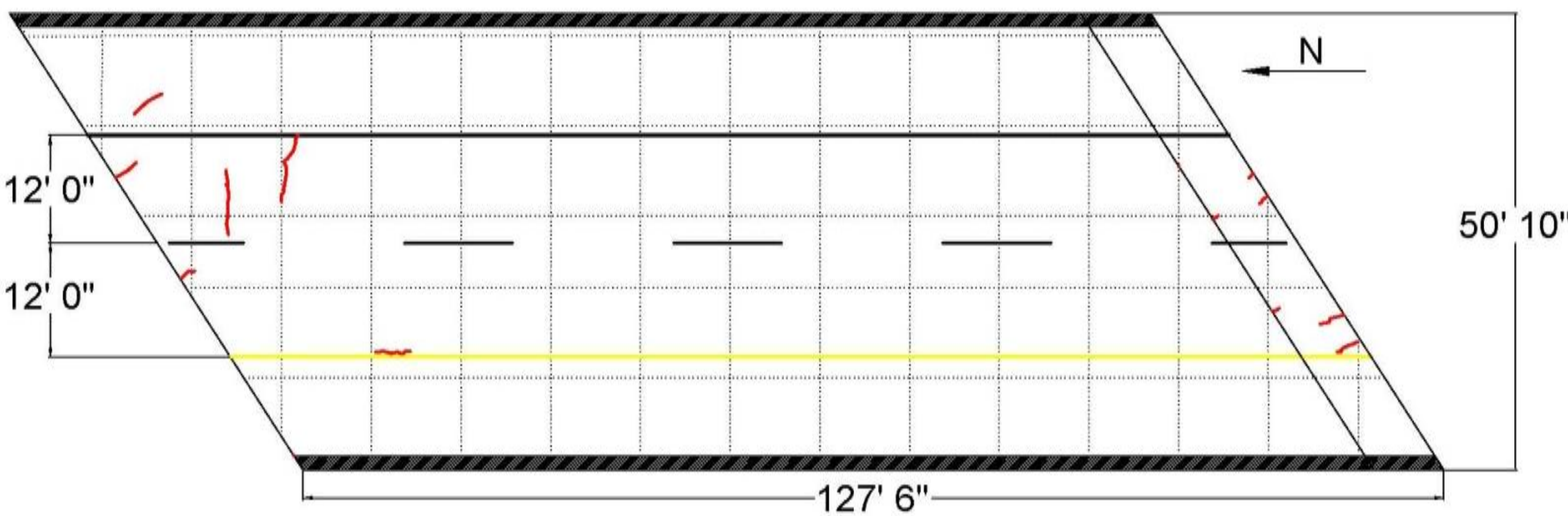

Figure C-16 Distress map for Dannon Way northbound internally cured concrete deck at 8 months.

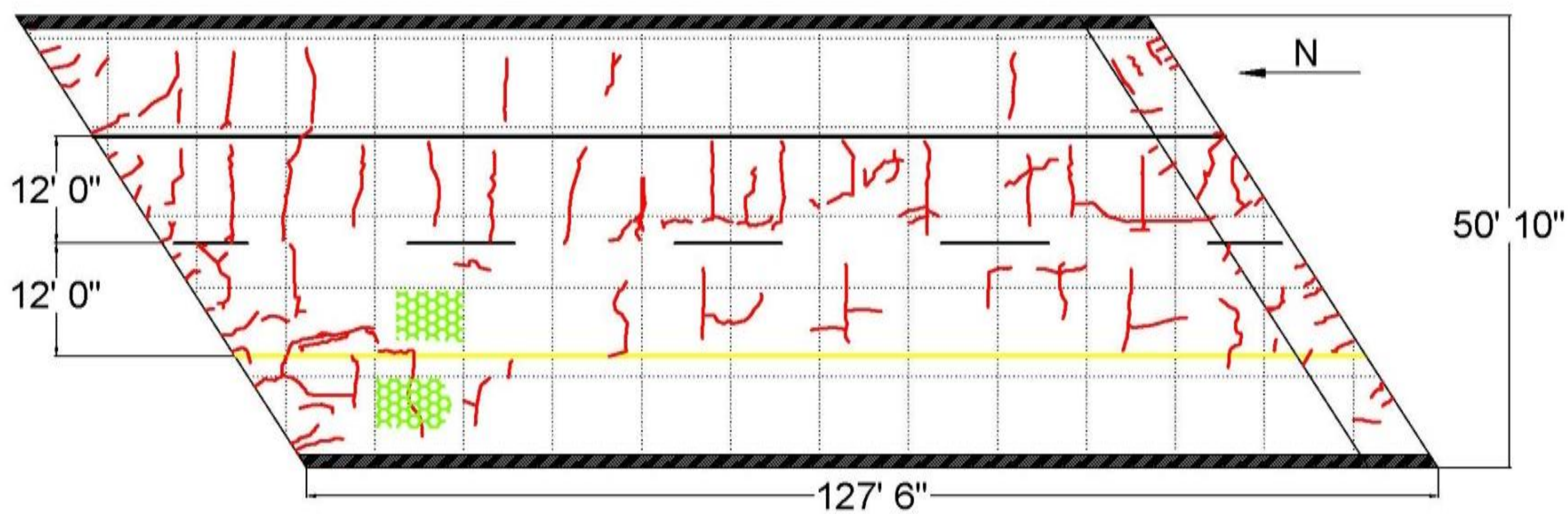

Figure C-17 Distress map for Dannon Way northbound internally cured concrete deck at 1 year.

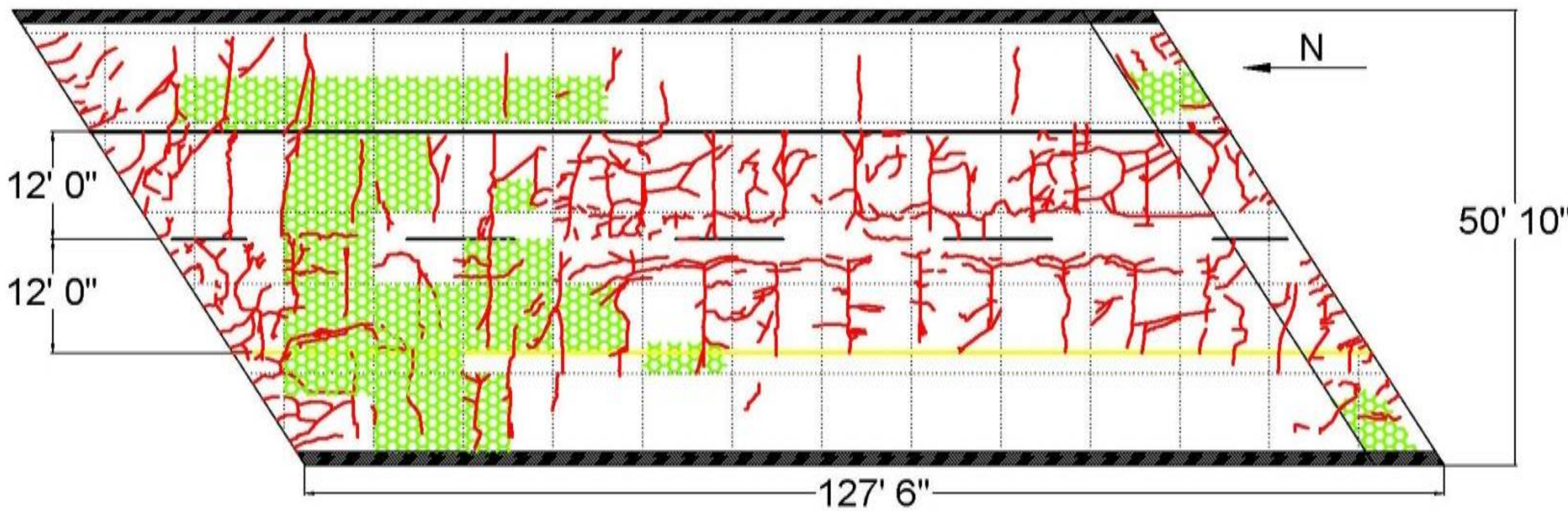

Figure C-18 Distress map for Dannon Way northbound internally cured concrete deck at 2 years.

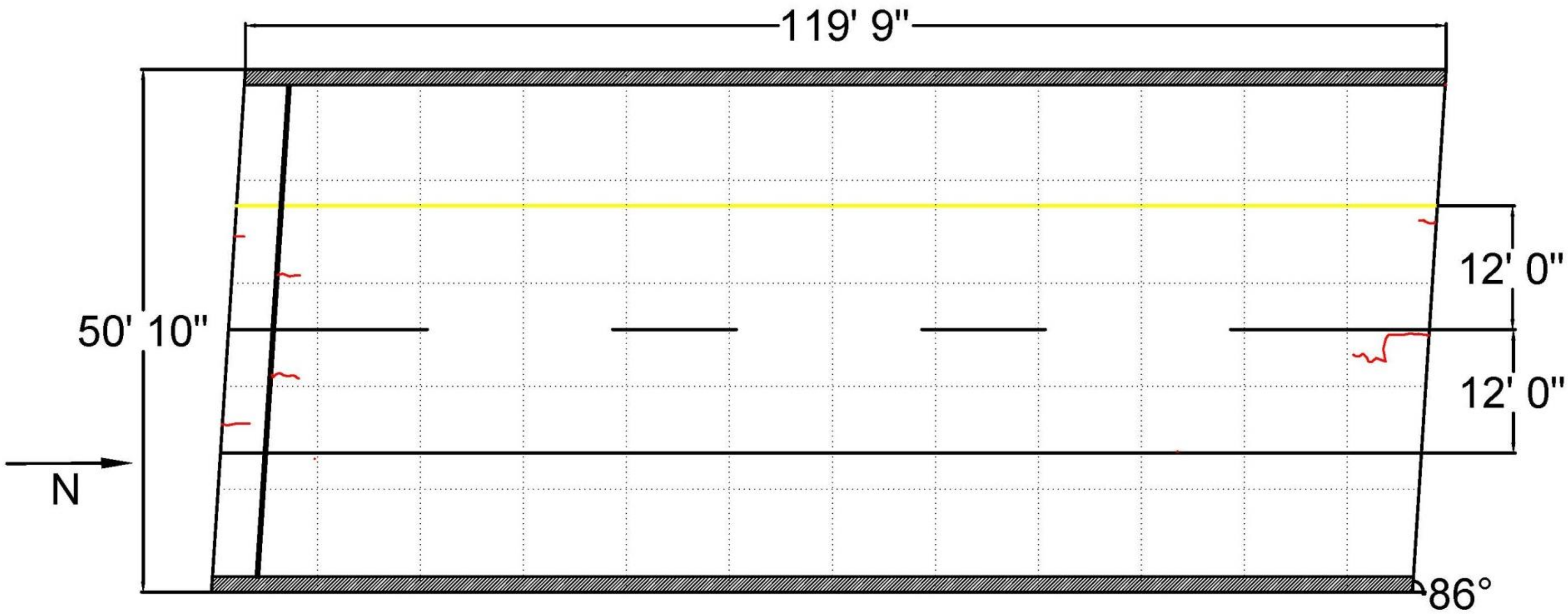

Figure C-19 Distress map for 8200 South northbound conventional concrete deck at 5 months.

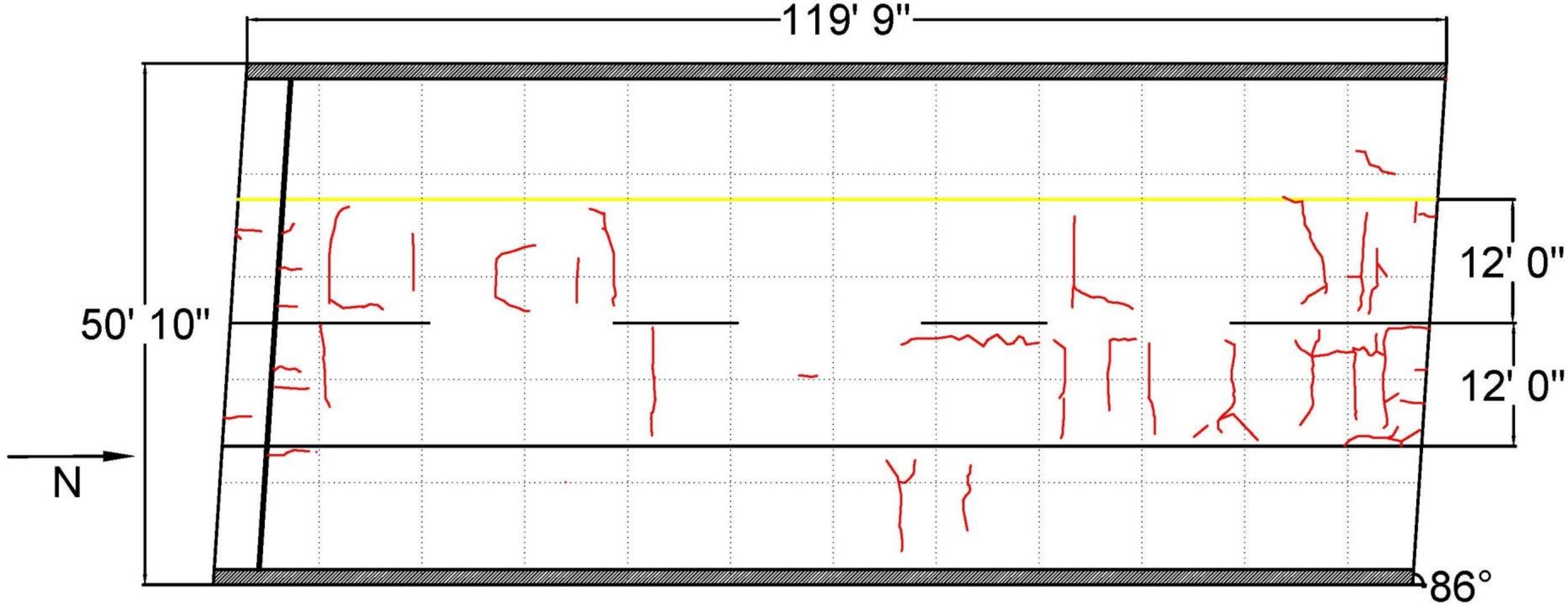

Figure C-20 Distress map for 8200 South northbound conventional concrete deck at 8 months.

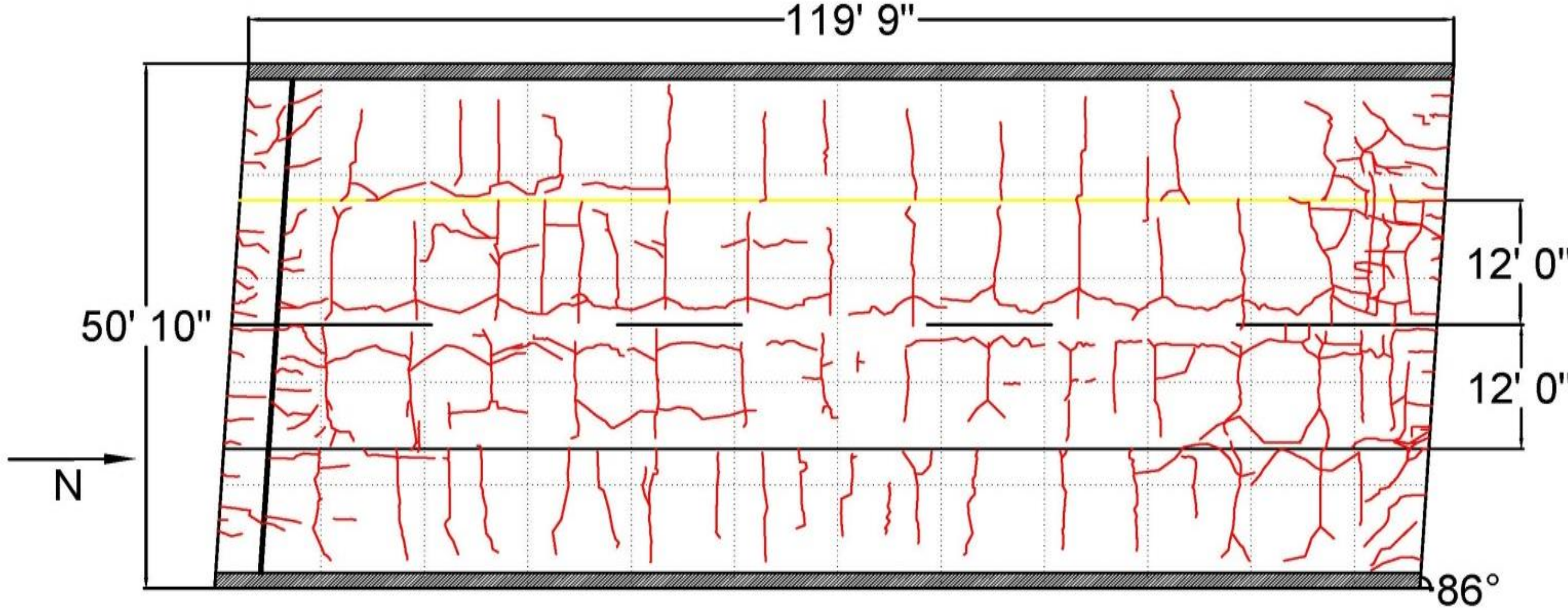

Figure C-21 Distress map for 8200 South northbound conventional concrete deck at 1 year.

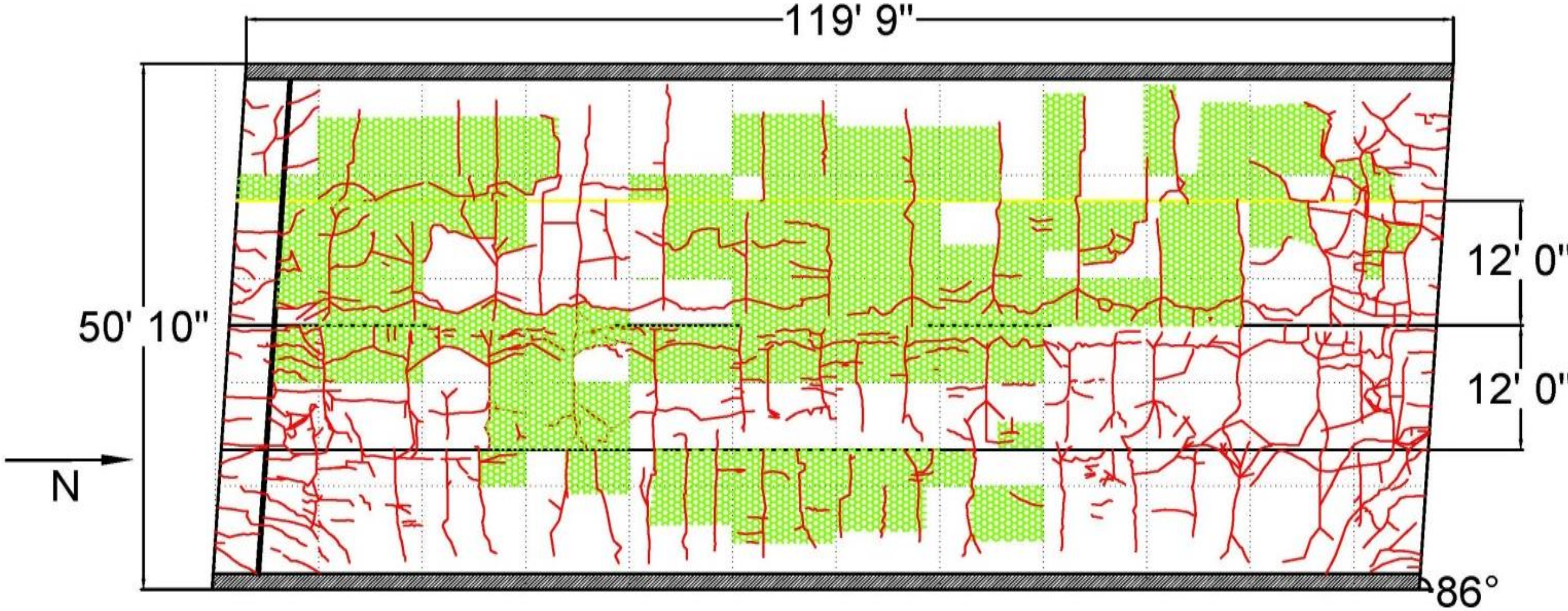

Figure C-22 Distress map for 8200 South northbound conventional concrete deck at 2 years.

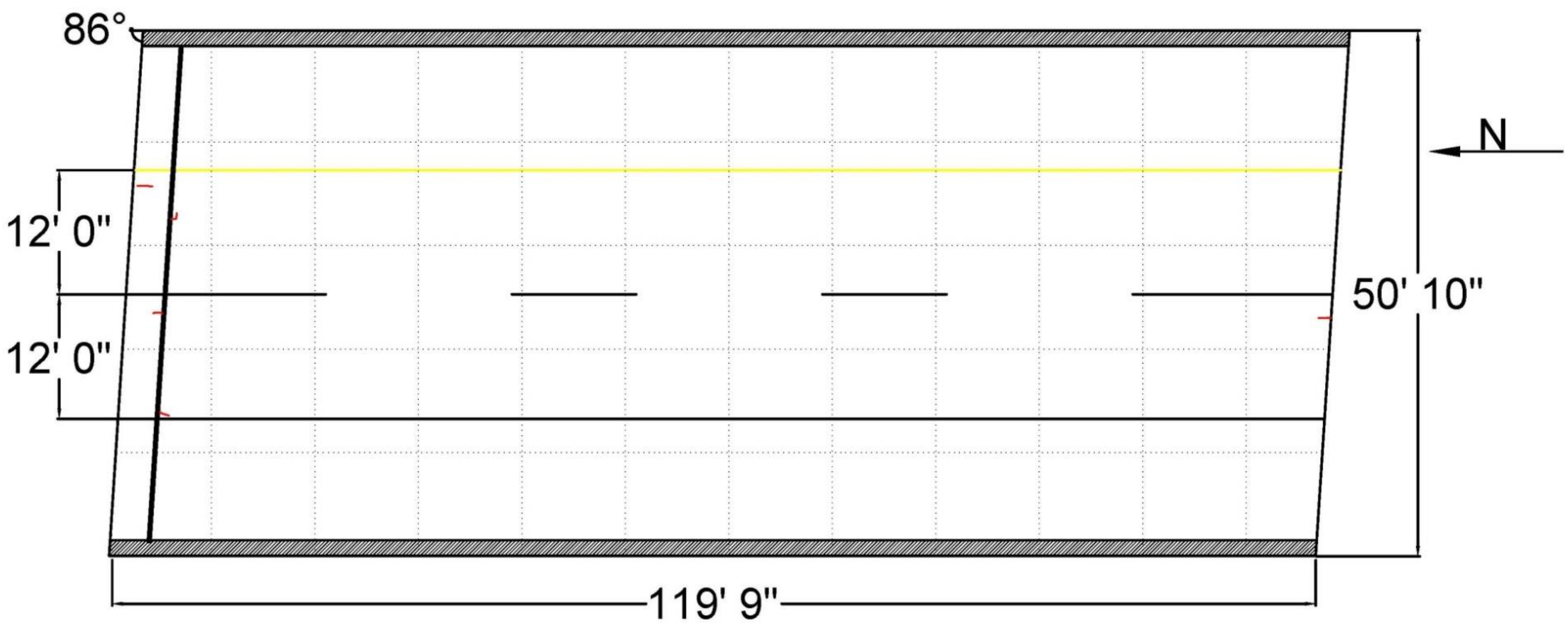

Figure C-23 Distress map for 8200 South southbound internally cured concrete deck at 5 months.

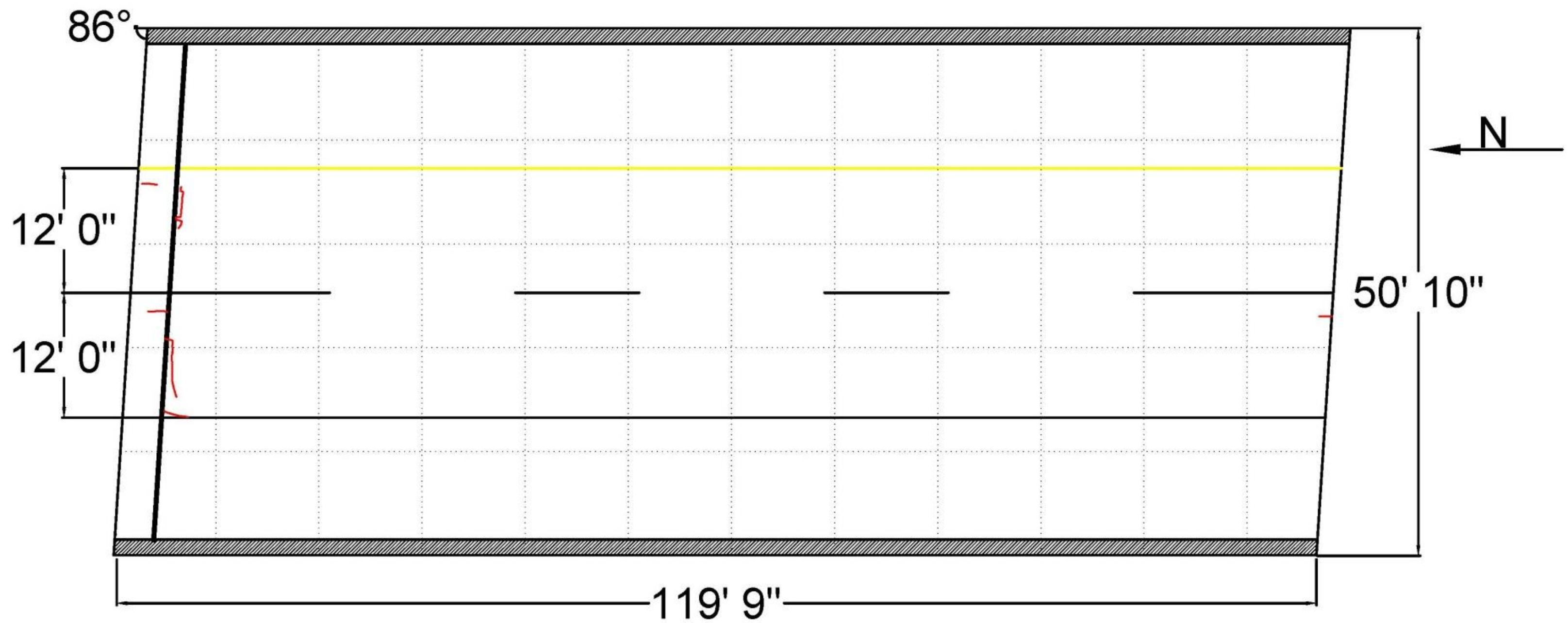

Figure C-24 Distress map for 8200 South southbound internally cured concrete deck at 8 months.

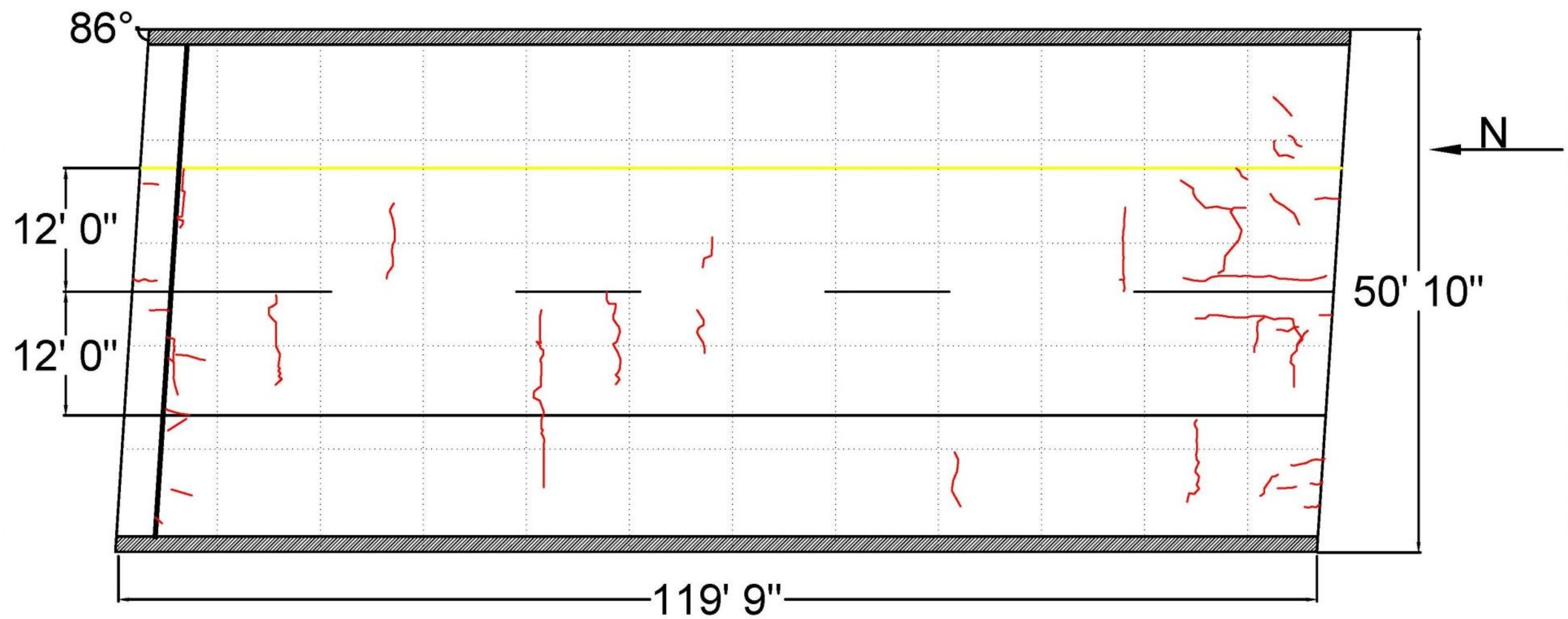

Figure C-25 Distress map for 8200 South southbound internally cured concrete deck at 1 year.

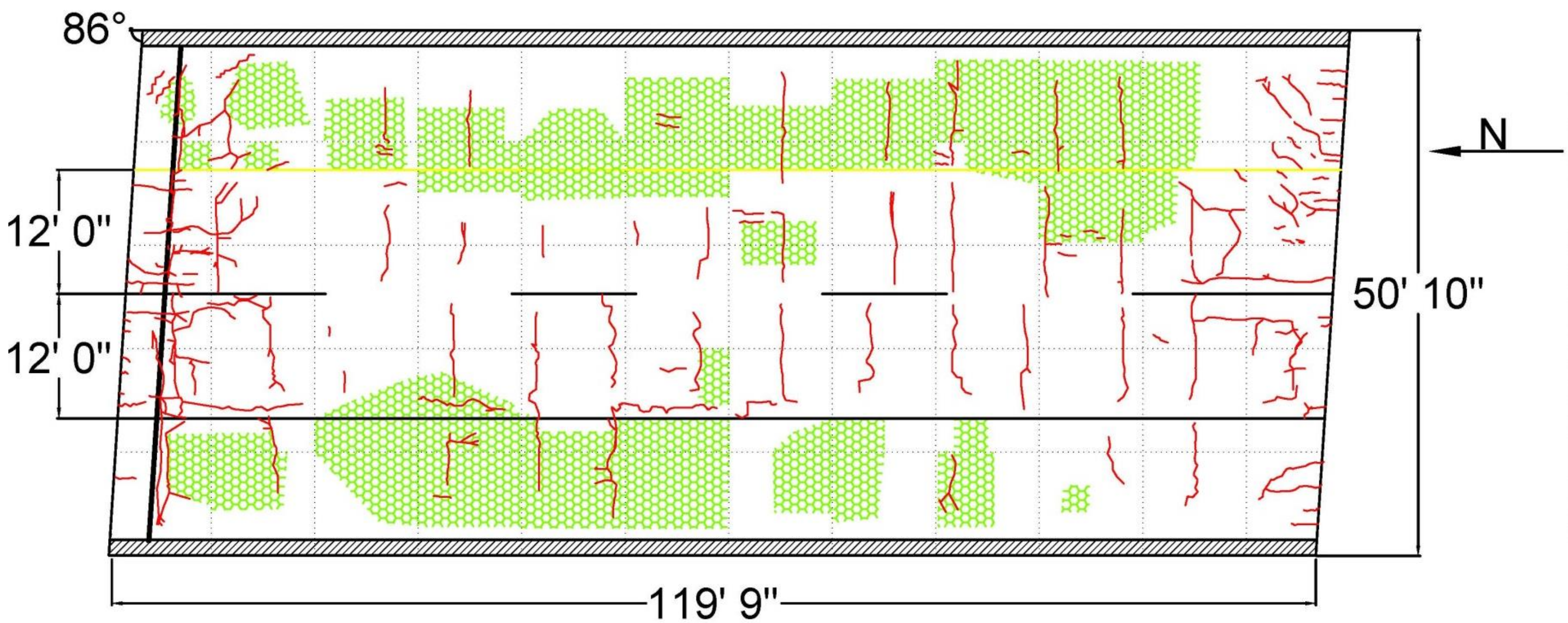

Figure C-26 Distress map for 8200 South southbound internally cured concrete deck at 2 years.