Subsidence Cracking of Concrete Over Steel Reinforcement Bar in Bridge Decks

By

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ABSTRACT

It is known that subsidence cracking may cause premature deterioration of concrete slab structures in salt laden environments. Chlorides from either deicing salts or marine environments may cause chloride-induced corrosion of the reinforcing steel resulting in spalling of the cover concrete. Concrete specimens with 16 mm (# 5) diameter bars were cast with various cover depths, bar spacing and two concrete mixture types to determine the influence that epoxy coated reinforcement, cement type and bar spacing may have on the probability of subsidence cracking in bridge deck slabs. It was determined that there is not a significant difference in the probability of cracking of concrete between concrete cast with epoxy coated reinforcing steel and bare reinforcing steel. Concrete subsidence cracking was found to be dependent upon the clear cover depth and cement type.

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1. INTRODUCTION

The function of bridge decks are to distribute dead and live loads, increase the bending capacity of girders (composite construction), and provide a resistant wearing surface. This structural component is in direct contact with harsh environmental degradation mechanisms such as freezing and thawing and corrosion from deicing salts. The primary cause of bridge deck deterioration is corrosion of the reinforcing steel (Weyers et al., 1982).

Cady and Weyers (1984) presented a service life model predicting the effective service life of reinforced concrete members when chloride-induced corrosion is the primary cause for deterioration. The service life model consists of four separate time periods: rapid initial damage resulting from construction faults (subsidence cracking), chloride diffusion period to the least 2.5 percentile bar cover depth, corrosion of the steel of the 2.5 percentile reinforcing bar until cracking occurs, and subsequent uniform deterioration to level of damage defined as the end-of functional service life (Weyers, 1998). The above service live model time periods are illustrated in Figure 1-1.

In concrete, reinforcing steel is protected from corrosion by the high pH environment afforded by the high concentration of hydroxides in the concrete pore water solution. Water-soluble chloride may, however, penetrate to the level of the steel in sufficient quantities to destroy the passivity of the steel. Water-cement (w/c) ratio, degree of consolidation, and depth of concrete cover over the reinforcing steel may affect the degree of penetration of the chloride ion in uncracked concrete (Weyers et al., 1983).

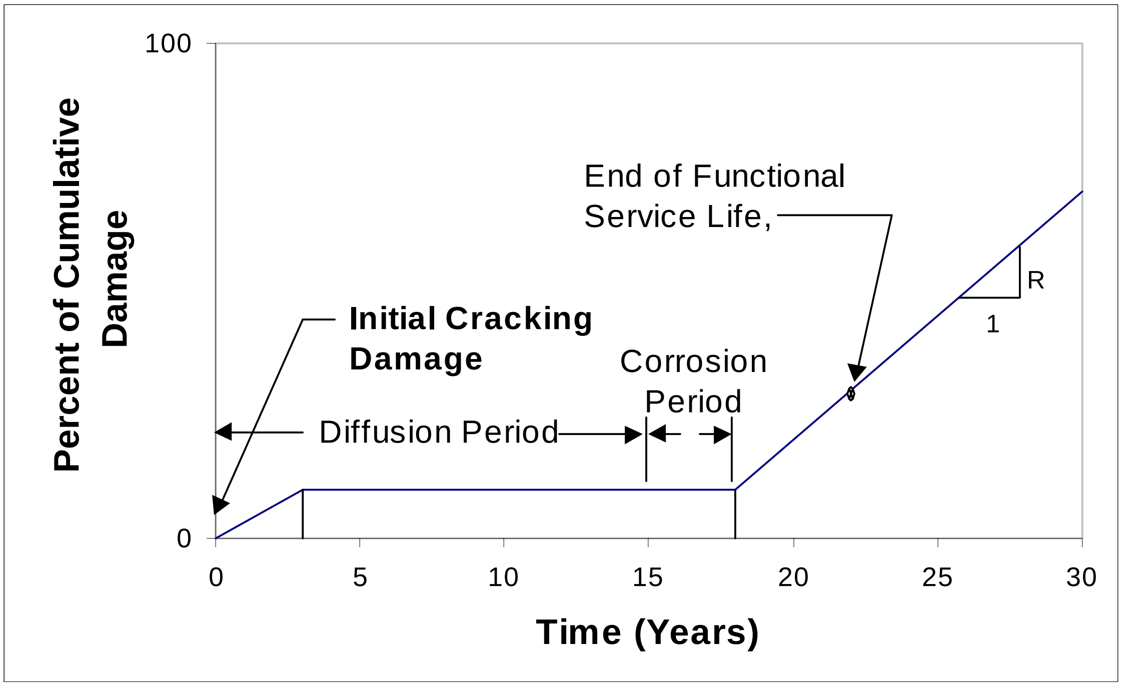


Figure 1-1: Concrete Service Life Model Relative to Corrosion Deterioration (Weyers, 1998)

It is believed that the first stage in deterioration is the formation of cracks above the reinforcement bar as a result of a combination of design parameters and construction methods. Plastic shrinkage, dry shrinkage, thermal shrinkage, autogenous shrinkage, bending and subsidence stresses may singularly or in concert crack the cover concrete. These cracks resulting from design and/or construction practices provide a direct means for water, oxygen and chloride ions from deicing salts to penetrate to the steel reinforcement, initiate corrosion and spall the cover concrete.

1.1 Early Stage Bridge Deck Cracking

Initial bridge deck cracking occurs as a result of tensile stresses (shrinking and bending) that take place in the concrete. Bridge deck cracks typically run transversely across the bridge deck (Krauss and Rogalla,1996). Early age concrete shrinkage (less than 24 hr after placement) is of concern because the concrete has not gained adequate strength to resist the imposed tensile stresses. Concrete at an early stage has a low tensile capacity and therefore the highest risk of cracking (Holt and Janssen, 1997). Horizontal restraint of concrete by longitudinal girders may also have a significant impact on the amount of transverse cracking. The imposed tensile stresses found in concrete bridge decks may be the result of plastic shrinkage, thermal shrinkage, dry shrinkage, autogenous shrinkage, bending stresses and concrete subsidence.

1.1.1 Horizontal Deck Restraint

Typical bridges today are designed as composite where the concrete deck serves as a wide flange for the composite cross section. In many cases, bridge deck concrete shrinkage is not compatible with the prefabricated steel or concrete girders. For this reason, concrete decks may crack due to horizontal restraint afforded by reinforcing steel and/or longitudinal shear studs attached to the girder. Deep steel-girder bridges are the most susceptible to deck cracking due to restrained shrinkage (Krauss and Rogalla, 1996).

1.1.2 Plastic Shrinkage

Plastic shrinkage results from the evaporation of surface moisture during the initial and final concrete setting periods. Plastic shrinkage takes place when the evaporation rate becomes greater than the rate at which bleed water rises to the surface.

The rate of evaporation of bleed water is a function of concrete temperature, air temperature, wind speed and relative humidity (Lerch, 1957).

1.1.3 Thermal Shrinkage

As concrete cures, the concrete temperature will elevate during the release of the heat of hydration. During the process of cooling to ambient temperatures, thermal shrinkage of the hardened concrete may induce tensile stresses in the concrete. Differential cooling of concrete bridge decks aggregated by horizontal girder restraints, may allow the tensile stresses to become larger than the tensile strength of the newly cast concrete, resulting in longitudinal and transverse cracking.

1.1.4 Drying Shrinkage

Drying shrinkage is another common cause of construction-related cracking. Drying shrinkage results from the loss of original concrete mix water after the concrete has cured. As free water in the concrete pore structure evaporates, the concrete shrinks. As in thermal shrinkage, transverse cracking due to drying shrinkage may be aggravated by longitudinal restraints afforded by the girders. These cracks tend to follow in line with the top mat of reinforcement bars and are full depth (Babaei and Fouladgar, 1997).

1.1.5 Autogenous Shrinkage

Autogenous shrinkage may cause adverse volume changes in concrete. The magnitude of autogenous shrinkage is governed only by the material chemistry and internal binding structure of the concrete mixture. More specifically, the cement fineness and concrete admixtures may significantly influence autogenous shrinkage (Holt and Janssen, 1997).

1.1.6 Bending Stresses

Dead and live loads may also induce flexural stresses causing cracking in the negative moment regions along the span. Flexural cracking of this nature usually takes place transversely above the superstructure (beams and girders). Cracking due to bending stresses may be reduced by placing the concrete at the center of the span followed by placing the concrete at the supports. This type of cracking may occur during unshored construction practices or from general service loads (Babaei and Fouladgar, 1997).

1.1.7 Concrete Subsidence

Finally, subsidence cracking may take place on bridge decks during the construction phase as a result of differential concrete settlement. Immediately following consolidation and finishing of the concrete, subsidence cracks may form on horizontal surfaces (bridge decks) while the concrete is in a plastic state. Subsidence cracks result from the concrete mixture subsiding around rigid inclusions such as horizontal reinforcement bars. During concrete subsidence, bleed water rises to the surface and the overall volume of the concrete is reduced. Cracks typically form above and parallel to the top reinforcing bars, which are transverse to the longitudinal deck beams or girders. The formation of subsidence cracks is demonstrated in Figure 1-2.

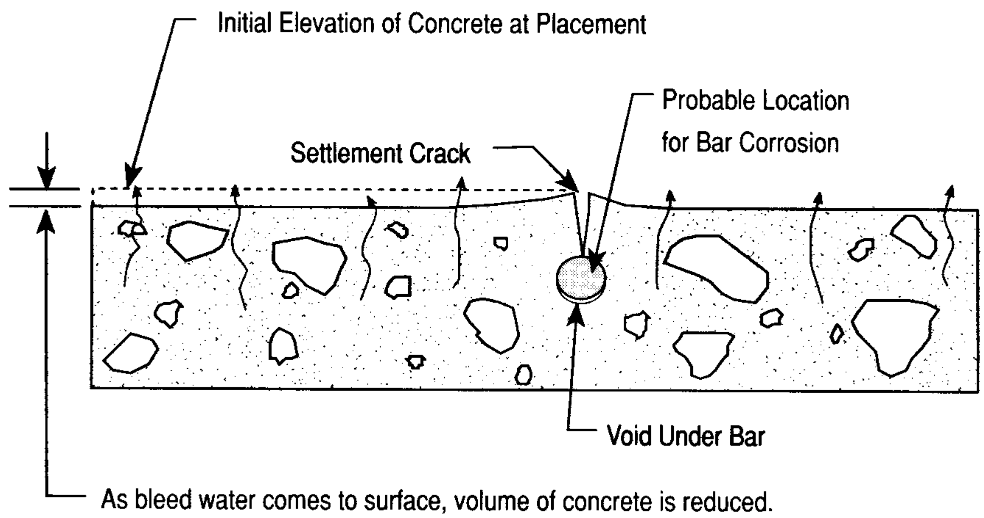


Figure 1-2: Subsidence Cracking of Concrete over Steel Reinforcement. (Emmons, 1993)

Several researchers have made efforts to understand the mechanics and behavior of subsidence cracking. Dakhil et al. (1975) showed that the probability of subsidence cracking in concrete bridge decks is a function of three primary factors: concrete clear cover, concrete slump and reinforcement bar size. In addition,

Weyers et al. (1982) showed that concrete subsidence cracking may be influenced by the bar spacing and consistency of the mixture. Concrete settlement was also shown to be influenced by concrete boundary conditions (form sides) (Powers, 1939). The following Literature Review section presents a more detailed discussion of the past research conducted on subsidence cracking. Previous studies have identified five primary factors that may influence subsidence cracking. These factors are:

- Clear cover depth: Shallow cover depths may greatly increase the occurrence of subsidence cracks.
- Concrete consistency (slump): Concrete that has a high slump may increase the occurrence of subsidence cracks.

- Reinforcing bar size: The use of large diameter reinforcement bars may increase the occurrence of subsidence cracks.
- Reinforcement bar spacing: Concrete slabs with reinforcement bars spaced farther apart may exhibit a greater probability of subsidence cracking.
- Concrete boundary conditions: Concrete settlement capacity may be influenced by boundary friction effects at and between boundary points.

Within current bridge design specifications, concrete subsidence alone may not cause cracking. Even if subsidence cracks do not appear, vertical planes of weakness may form parallel to and above reinforcement bars (Babaei and Fouladgar, 1997). These planes of weakness are caused by the initial settlement-induced strains of the plastic concrete around the bar. Cracking may occur above transverse reinforcement bars from complementary stresses induced from drying, thermal shrinkage and/or bending. This might explain why transverse cracking has been detected in recently constructed bridge decks during field surveys (Babaei and Fouladgar, 1997).

1.1.8 Significance of Crack Width

Most cracks found in concrete bridge decks are aligned with the steel reinforcement bar exposing large surface areas of the bar. It is for this reason that crack width is as important as the frequency of cracking. The National Cooperative Highway Research Program Report 380 reported that crack widths 0.05 mm (0.002 in.) or wider may allow infiltration of salt contaminants (Krauss and Rogalla, 1996).

2. LITERATURE REVIEW

2.1 Concrete Subsidence Studies

Early studies of subsidence cracking were conducted by Dakhil et al. (1975). They quantified the influence of cover depth, concrete slump and reinforcement bar size has on the probability of the occurrence of subsidence cracking. Concrete with a water to cement ratio (w/c) of 0.48 was utilized to show that subsidence cracking is controlled primarily by cover depth, concrete slump and reinforcement bar size. These results were supported by additional studies employing the use of a photoelastic material (gelatin) to model concrete subsidence specimens (Dakhil et al., 1975). From their model, it was shown that the maximum stress induced by subsidence is tensile, occurring tangentially over the reinforcement. Maximum tensile stresses varied from 0.002 MPa to 0.008 MPa (0.3 to 1.2 psi) for cover depths ranging from 19 to 51 mm (0.75 to 2 in.). This tensile zone was found to extend ± 25.4 mm (± 1 in.) from the vertical centerline of the bar, where a sign reversal occurred, resulting in tangential compressive stresses that extended at a nearly constant value out to the edge of the concrete specimens (Dakhil et al., 1975).

In addition, Dakhil et al. (1975) showed that the corrosion current density is much greater for cracked specimens than uncracked specimens in salt-laden environments. This showed that cracked specimens have a greater potential for corrosion than uncracked specimens. It was concluded that spalling, the most serious form of bridge deck deterioration is significantly influenced by inadequate reinforcement bar cover depth.

Another laboratory study was conducted by R.E. Weyers et al. (1982). The study demonstrated the influence of inclusion spacing on the probability of occurrence of subsidence cracking. A photo elastic material, gelatin, was used to model the body force stresses generated during the settlement of fresh concrete. In modeling typical bridge deck conditions, 16 mm diameter (# 5) reinforcement bars and typical bar spacings of 76.2, 114.3, and 152.4 mm (3, 4.5, and 6 in.) were used in the study. It was shown that the tensile stresses above the bar may be related to the bar spacing, cover depth and the consistency of the mix. Of primary importance, it was demonstrated that the maximum tensile stresses above the reinforcement bar decreased with increasing cover depths and that the tensile stresses formed during concrete subsidence may be of sufficient magnitude to produce surface cracking in the plastic concrete. It was concluded that differential settlement of concrete around reinforcing bars may significantly affect the time to corrosion deterioration in bridge decks (Weyers et al., 1982).

In addition, Weyers et al. (1982) demonstrated through photoelastic studies in a gelatin medium that various boundary friction effects reduce the settlement capacity of the concrete at and between boundary points. This supported earlier studies conducted by Powers (1939) on the influence of boundary friction effects on concrete. Powers showed that the concrete settlement capacity is a function of column height, water to cement ratio (w/c) and slump. It was determined that maximum settlement occurs within 90 to 100 minutes after concrete consolidation for column heights greater than 120 mm (4.75 in.). It was also shown that concrete setting often begins before the concrete has fully subsided. Points located near boundary conditions were found to settle at reduced rates as compared to points free from boundary condition restraints. For a maximum bond between the concrete and reinforcement bar along the underside of the steel reinforcement, Powers recommended using concrete mixtures with the lowest slump practicable and using cement paste with a low bleeding capacity. He showed that concrete with slump values greater than 0.7 in. would exhibit bleeding.

Two studies have been conducted to develop a better understanding of shear strength development in concrete mixtures. The first study, conducted by Olson (1968), studied the development of shear strength in fresh concrete using undrained triaxial tests. He found that during early hydration periods, the shear strength of fresh concrete consists primarily of cohesive bonding of the cement paste. As the early hydration period increases, the cement paste becomes less plastic and the mobility of the aggregate particles decrease, resulting in an increasing angle of internal friction.

The second study, conducted by Alexandridis and Gardner (1981), compared fresh concrete to soil; concrete is a particulate system composed of particles weakly bonded together and submerged in a liquid medium. Fresh concrete may transmit forces through the voids (pore pressure), particle contact points, and through the cross-section of the particles. This study which was similar to that by Olson (1968) included the effects of pore pressure in the concrete matrix. Alexandridis and Gardner (1981) showed that shear strength in fresh concrete resulted from internal friction between interlocking aggregate, cement particles, and the bonding together or cohesion between the aggregate and cement paste during hydration. They further explained that true cohesion can take place only as a result of cement hydration.

Alexandridis and Gardner (1981) concluded that cohesion depends on the time after the addition of water and hydration temperature. In addition, they offer a model for concrete shear strength development that is slightly different than the one presented by Olson. Immediately following mixing, the shear strength of fresh concrete exists from internal friction resulting from particle interaction between concrete voids and aggregate particles. Additional development of shear strength in the concrete results primarily from the development of cohesion bond afforded by the cement hydration process. At lower temperatures, hydration rates are retarded and shear strength from cohesion must develop over a longer period of time. Thus concrete mixtures at low temperatures will behave more like a fluid for extended time periods as compared to concrete cured at elevated temperatures.

2.2 Current Controls over Subsidence Related Stresses

Concrete bridge decks today are expected to be highly durable, having little to no initial cracks. Virginia Department of Transportation (VDOT) specifications have been written prescriptively to limit bridge deck damage caused by construction-related activities (VDOT Road and Bridge Specifications, 1997).

Bridge deck design specifications in Virginia require a w/c ratio at or below 0.45, the use of # 57 non-polishing coarse aggregate (nominal 25 mm (1in.)), and an allowable slump of 51 to 100 mm (2 to 4 in.) or 175 mm (6.5 in.) for water-reduced concrete. Concrete cover depths are required to be greater than or equal to 65 mm (2.5 in.), with a tolerance within 0 to + 13 mm (0 to + 0.5 in.). Virginia bridge deck concrete is to have an air content of 5 – 7% and a minimum 28-day compressive strength of 27.6 MPa (4000 psi) (VDOT Road and Bridge Specifications, 1997).

In addition to the above specifications, VDOT Road and Bridge Specifications provide the following guidelines for construction practices. The concrete is required to be consolidated with a mechanical vibrator internally such that the intensity of vibration is adequate to consolidate concrete thoroughly without causing segregation or excessive bleed water. As previously presented, the degree of initial consolidation affect the rate and degree of concrete subsidence. Other applicable excerpts concerning VDOT design and construction specifications are presented in Appendix III.

Babaei and Fouladgar (1997) offer other design recommendations to minimize the possibility of subsidence-related cracking over transverse reinforcing bars. They recommend not aligning transverse top and bottom mat reinforcement bars together. Top and bottom bars should be alternated and the use of large top transverse bars should be discouraged; top transverse bars should be limited to 16 mm (# 5) diameter.

3. SELECTION OF TEST PARAMETERS

3.1 Concrete Cover Depth

Dakhil et al. (1975) showed that concrete cover depth is the most critical design parameter related to subsidence cracking. The concrete cover depths chosen for this study were based upon the results of a 1998 study of in-service bridge decks (Pyc, 1998).

A study of 18 bridges showed that cover depths were normally distributed having an average cover depth of 65 mm (2.6 in.) with a standard deviation equal to 9.1 mm (0.358 in.) (Pyc, 1998). It was, however, shown that 8 of the 18 bridges had individual average cover depths under 65 mm (2.6 in.). In addition, the span with the lowest 12 percentile cover depth was determined for each bridge deck. Of the 18 bridge decks, 7 had a 12 percentile cover depth under 50 mm (2.0 in.); 8 of the decks ranged between 51 and 65 mm (2.0 and 2.6 in.) and only 1 bridge deck had a lowest 12 percentile cover depth of 66 mm.

The significance of the 12 percentile concrete cover depth was shown by Fitch et al. (1995). They determined from a number of survey questionnaires that damage to 12 percent of the worst deteriorated traffic lane defined the end of function service life. The survey questionnaires taken in the study included inspection of visual deterioration conditions of bridge decks by a number of bridge engineers.

Cover depths of 38, 44, 51, 57 and 64 mm (1.5, 1.75, 2, 2.25, 2.5 in.) were chosen for this study to test the range of cover depths observed in Virginia bridge decks. The cover depths chosen range between 0 to 3 standard deviations from the average cover depth of 65 mm (2.5 in.). A concrete cover depth of 38 mm (1.5 in.) corresponds to 3 standard deviations (65 – (3)*(9.1)) from the average cover depth. The other cover depths of 44, 51, 57 and 64 mm (1.75, 2, 2.25, 2.5 in.) were chosen to test for situations in which cover depths were less than 3 standard deviations from the mean. Statistically, this range of cover depths would test all but 0.13 percent of the steel observed in the field.

3.2 Specimen Design Size

Previous studies conducted by Dakhil et al. (1975) used 305 mm (12 in.) square sample sizes. The design specimen sizes (813, 610 and 305 mm square) were chosen to provide a range of boundary conditions to determine the role reinforcement bar intersections and form edges may have on subsidence cracking. In addition, the 305 mm (12 in.) specimens were cast to provide a relative comparison to research conducted by Dakhil et al. Specimen sizes 813 and 610 mm were chosen to provide a comparison between 150 and 200 mm (6in. and 8 in.) bar spacing. The specimen depth was set at a constant 208 mm (8 in.) to simulate typical bridge deck design thicknesses found in Virginia bridge decks.

3.3 Bar Spacing

For this study, concrete reinforcement bar spacing was chosen to be 152 and 203 mm (6 in. and 8 in.). Reinforcement bar spacing requirements for Virginia bridge decks are governed by AASHTO (1996) design requirements. Bar spacing for primary reinforcement (transverse bars) must be less than or equal to the minimum of 1.5 times the slab thickness or 450 mm (18 in.). The minimum bar spacing is limited to the greater of 1.5 bar diameters, 1.5 times the maximum aggregate, or 38 mm (1.5 in.). Bar spacing of 152 and 203 mm (6 and 8 in.) were selected as being typical for bridge deck design.

3.4 Reinforcing Bar

In this study, 42 specimens were cast with epoxy coated reinforcement (ECR) and 42 were cast with bare reinforcement (BRS). Specimens cast with BRS served as the test control necessary for determining the influence epoxy coating may have on subsidence cracking. Efforts were made to limit effects caused by geometrical differences in reinforcing bars. This was achieved by using steel bars with the same deformation pattern. In addition, reinforcement bars were placed into the forms in the same radial orientation, every bar was rotated such that the longitudinal spine or rib was oriented upward. 16 mm (# 5) reinforcing bar was selected as being typical of bridge deck design.

3.5 Concrete Mixture Design

Previous subsidence crack studies done by Dakhil et al. (1975) were conducted with bare reinforcing steel (BRS) and Type I portland cement. Little information however exists relative to the use of ECR and blended cement concrete mixtures. To simulate field conditions, concrete mixture proportions were chosen as typical VDOT Class 30 (A4) bridge design mixes. These concrete mixture designs were chosen to provide a comparison between portland cement (PC) and ground granulated blast furnace slag (GGBFS/PC) blended cement mixtures relative to subsidence cracking.

3.6 Concrete Protection

Cracking due to subsidence induced stress was the chief concern of this study. Efforts were made to limit other types of construction related stresses in the concrete. Specimens were outdoors and protected from direct exposure to sunlight and wind to reduce the probability of other types of cracking such as plastic and drying shrinkage. The surfaces of each concrete specimen were also kept wet to a sheen during the first 6 hours of curing to limit the effects of plastic shrinkage. Concrete specimens continued moist curing under burlap and plastic for three days. In addition, drying shrinkage effects were controlled to some degree by keeping the specimens out of direct sun for an additional 15 days. Other efforts were made to control shrinkage at the specimen boundary conditions, the plywood forms were left in-place.

3.7 VDOT Specifications

To better simulate actual field conditions in Virginia bridge decks, VDOT standard specifications were considered during the design, construction and protection of the test specimens. More specifically, VDOT requirements on concrete design parameters, control techniques, reinforcement, forms, consolidation, finishing and curing were considered in the formulation of this research plan. See Appendix III for applicable requirements set out by the 1997 VDOT Road and Bridge Specifications considered in this study.

Several deviations were made from typical design practices to assist specimen observations. Rather than finishing the concrete with a moist burlap drag, the final texture was finished with a magnesium float. This construction practice provided a relatively smooth concrete surface that assisted crack detection and measurement practices. In addition, concrete specimens were wet cured for three days only. Current VDOT Road and Bridge Specifications (1997) require a seven-day curing period. This deviation from the specification was designed to simulate possible adverse, in field, concrete bridge deck curing conditions found in Virginia.

4. PURPOSE AND SCOPE

This report conveys the results of laboratory studies conducted on subsidence cracking of VDOT Class 30 (A4) concrete over 16 mm diameter (# 5) reinforcing bars. The objectives of this study were to:

- Determine the effect epoxy coating on steel reinforcing bar may have on the occurrence of subsidence cracking of concrete bridge decks.
- Determine the effect that ground granulated blast furnace slag cement and portland cement concrete mixtures may have on the occurrence of subsidence cracking.
- Determine the degree of subsidence cracking of typical Class 30 (A4) concrete mixtures throughout the as-built bridge deck cover depths and typical bar spacing found in Virginia.
- Develop a comparison of the degree of subsidence cracking of concrete in specimens cast with a single reinforcing bar and multiple reinforcing bars.
- Assess the influence of drying shrinkage of concrete on further propagation of subsidence cracks over 16 mm diameter (# 5) reinforcement bars.

5. METHODS AND MATERIALS

5.1 Research Plan

A total of 84 concrete specimens were cast. The study included five cover depths, two bar spacings, and two concrete mixture types. The prisms were 208 mm (8 in.) thick. The three square surface dimensions were 813, 610 and 305 mm (32, 24 and 12 in.). Cover depths were 38, 44, 51, 57 and 64 mm (1.5, 1.75, 2.0, 2.25, and 2.5 in.) and the bar spacings were 150 and 200 mm (6 and 8 in.). Figure 5-1 through Figure 5-3 present a schematic diagram of the three prismatic types. A blended portland cement (PC) and ground granulated blast-furnace slag (GGBFS) cement concrete mixture (Batch 1) and a PC mixture were used (Batch 2 and 3). The reinforcing steel was 16 mm (# 5) in diameter; equal number of specimens were cast using epoxy coated (ECR) and bare reinforcement (ECR) bars. Six prisms were cast for each of the five cover depths, three with ECR and three with BRS. Table 5-1 presents the study test matrix.

Table 5-1: Subsidence Cracking Specimen Matrix

Specimen Surface Dimension	**813 x 813 mm**	**813 x 813 mm**	**610 x 610-mm**	**305 x 305-mm**
Bar Spacing	**203 mm**	**203 mm**	**152 mm**	**152 mm**
	Batch 1	**Batch 2**	**Batch 3**	**Batch 3**
Concrete Type	**60% PC 40% GGBFS**	**100% PC**	**100% PC**	**100% PC**
Number of Top ECR Bars	3	3	3	1
Cover Depths	Prisms	Prisms	Prisms	Prisms
38 mm (1.5 in.)	3	3	3	3
44 mm (1.75 in.)	3	3	3	3
51 mm (2.0 in.)	3	3	3	3
57 mm (2.25 in.)	-	-	-	3
64 mm (2.5 in.)	-	-	-	3
Number of Top BRS Bars	3	3	3	1
Cover Depths	Prisms	Prisms	Prisms	Prisms
38 mm (1.5 in.)	3	3	3	3
44 mm (1.75 in.)	3	3	3	3
51 mm (2.0 in.)	3	3	3	3
57 mm (2.25 in.)	-	-	-	3
64 mm (2.5 in.)	-	-	-	3

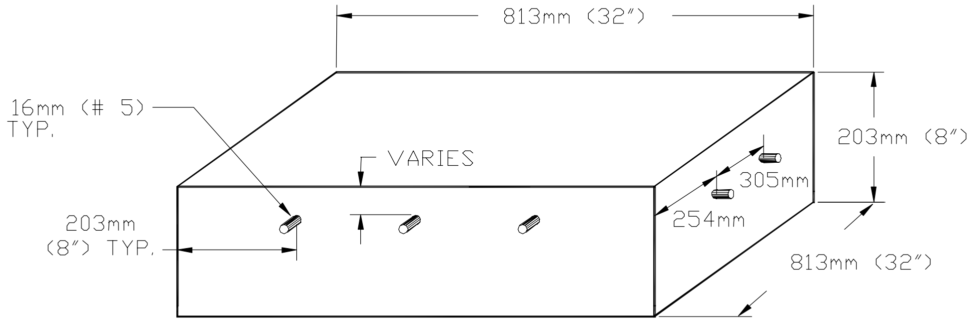


Figure 5-1a: Isometric Diagram for 813 mm (32 in.) Specimen Size

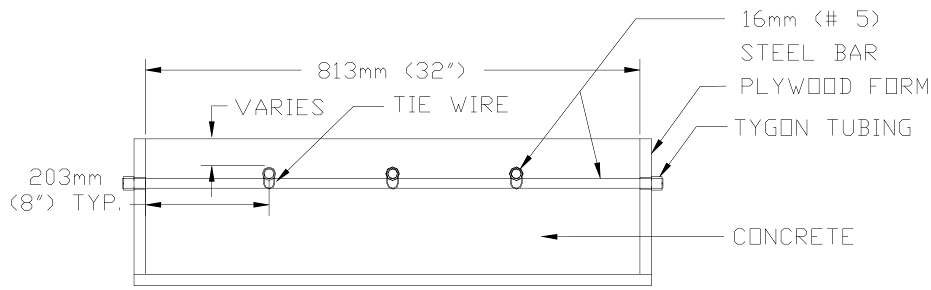


Figure 5-1b: Form and Specimen Diagram for 813 mm (32 in.) Specimen Size

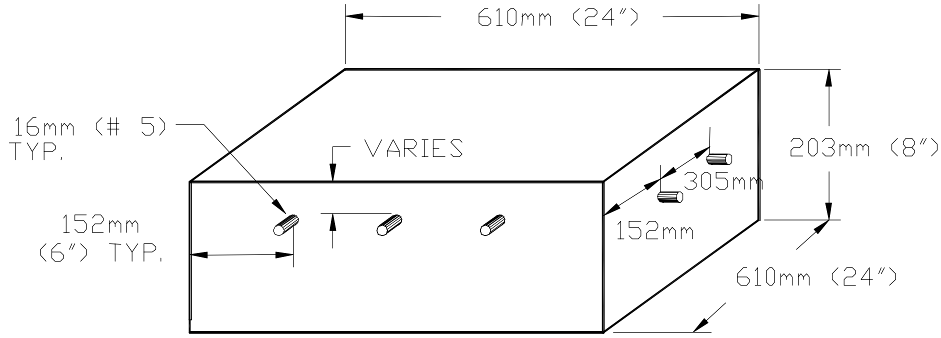


Figure 5-2a: Isometric Diagram for 610 mm (24 in.) Specimen Size

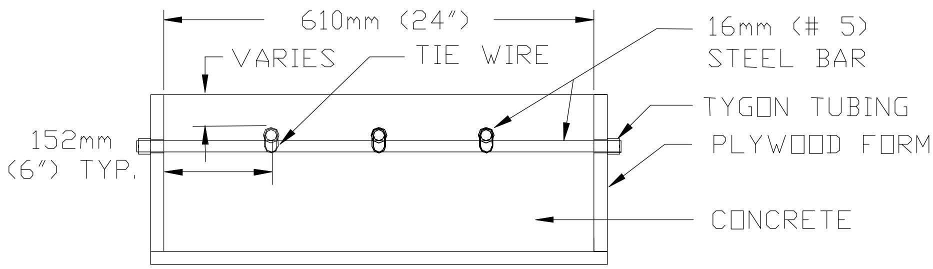


Figure 5-2b: Form and Specimen Diagram for 610 mm (24 in.) Specimen Size

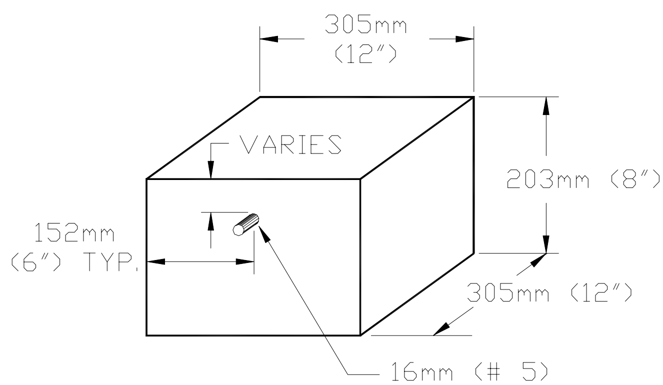


Figure 5-3a: Isometric Diagram for 305 mm (12 in.) Specimen Size

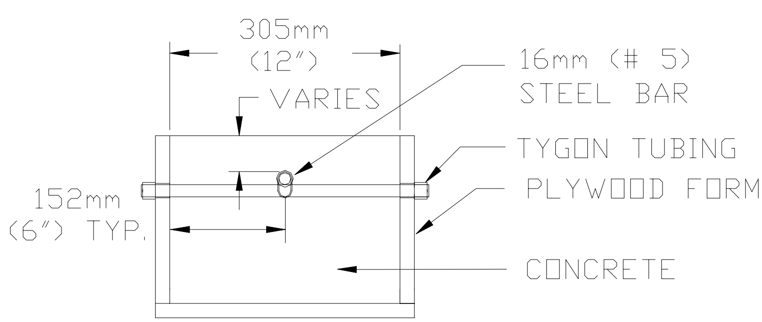


Figure 5-3b: Form and Specimen Diagram for 305 mm (12 in.) Specimen Size

5.2 Procedures

5.2.1 Concrete Mixture Design

As required by provisions set forth in Section 217.07 of the 1997 Virginia Department of Transportation (VDOT) Road and Bridge Specifications, VDOT Class 30 (A4) concrete mixture proportions were submitted by the concrete supplier and reviewed. Requirements for hydraulic concrete mixtures are given in VDOT Table II-17 entitled "Requirements for Hydraulic Cement Concrete"(pp. 195) of the specification. The concrete mixtures used in this study meet VDOT Section 217.07 of the 1997 VDOT Road and Bridge Specifications Manual.

5.2.2 Form and Steel Preparation

Steel reinforcement bars were cut from three-meter long bars using a band saw. For assembly reasons, bars were cut 76 mm (3 in.) longer than the concrete surface dimension. Care was taken to prevent coating damage to the ECR bars during specimen preparation.

Forms were constructed from 19-mm (3/4 in.) AC grade plywood. To provide the desired cover depths and bar spacing, 16-mm (5/8 in.) diameter holes were pre-drilled in the sides of the form. Wooden forms were coated with a concrete form oil before and after panel assembly. Reinforcing bars were held in with 16 mm (5/8 in.) diameter, 3 mm (1/8 in.) thick Tygon tubing placed on the exterior bar ends (See Figure 5). Bars were then tied at every bar intersection using standard uncoated reinforcing tie wire.

5.2.3 Concrete Placement, Consolidation, and Finishing

Concrete was placed on three separate days. Prior to placing, slump, concrete temperature, and air content measurements were taken to determine if the as delivered batches complied with VDOT Class 30 (A4) concrete specifications.

Once accepted, the ready mix truck concrete batches were placed in forms as a single layer using a concrete bucket and were consolidated with a 38 mm (1.5 in.) diameter electric vibrator (10,000 rpm). Concrete was screeded in the direction parallel to the reinforcing bar with a 2 x 4 board fitted with an electric vibrator and then finished using a magnesium float to facilitate the visual observations of the formation of subsidence cracks.

5.2.4 Concrete Curing Conditions

The concrete was placed outdoors and undercover to protect the concrete from direct exposure to sunlight and wind. The concrete prisms were wet cured in the forms under burlap for three days and kept out of direct sun for an additional 15 days.

5.2.5 Concrete Compressive Tests

Twelve 100 x 200-mm (4 in. x 8 in.) test cylinders were made for each concrete batch, six from the beginning and six from the end of the truck load. These cylinders were tested at various ages to determine the compressive strength development in the concrete over time. Table 5-2 shows the compressive test schedule used in this study. Two cylinders were tested at three and seven days, one cylinder from the beginning, and one from the end of the batch. In the same manner, four cylinders were tested at 28 and 182 days, two from the beginning and two from the end of each batch.

Table 5-2: Concrete Compressive Strength Schedule

	Batch 1		Batch 2		Batch 3	
Age, days	Front	Back	Front	Back	Front	Back
3	1	1	1	1	1	1
7	1	1	1	1	1	1
28	2	2	2	2	2	2
182	2	2	2	2	2	2

5.3 Evaluation Methods

5.3.1 Visual Subsidence Cracking Observations

Visual observations began almost immediately and every 30 minutes after the concrete specimens were consolidated and finished. Final initial observations and measurements were taken six hours after consolidation. The time to initial cracking, crack locations, crack length and crack width were recorded. Results were analyzed to determine the probability of subsidence cracking relative to concrete cover depth, reinforcement type and bar spacing, and type of concrete mixture used. For details concerning the results of the crack analysis see the Results section of this publication.

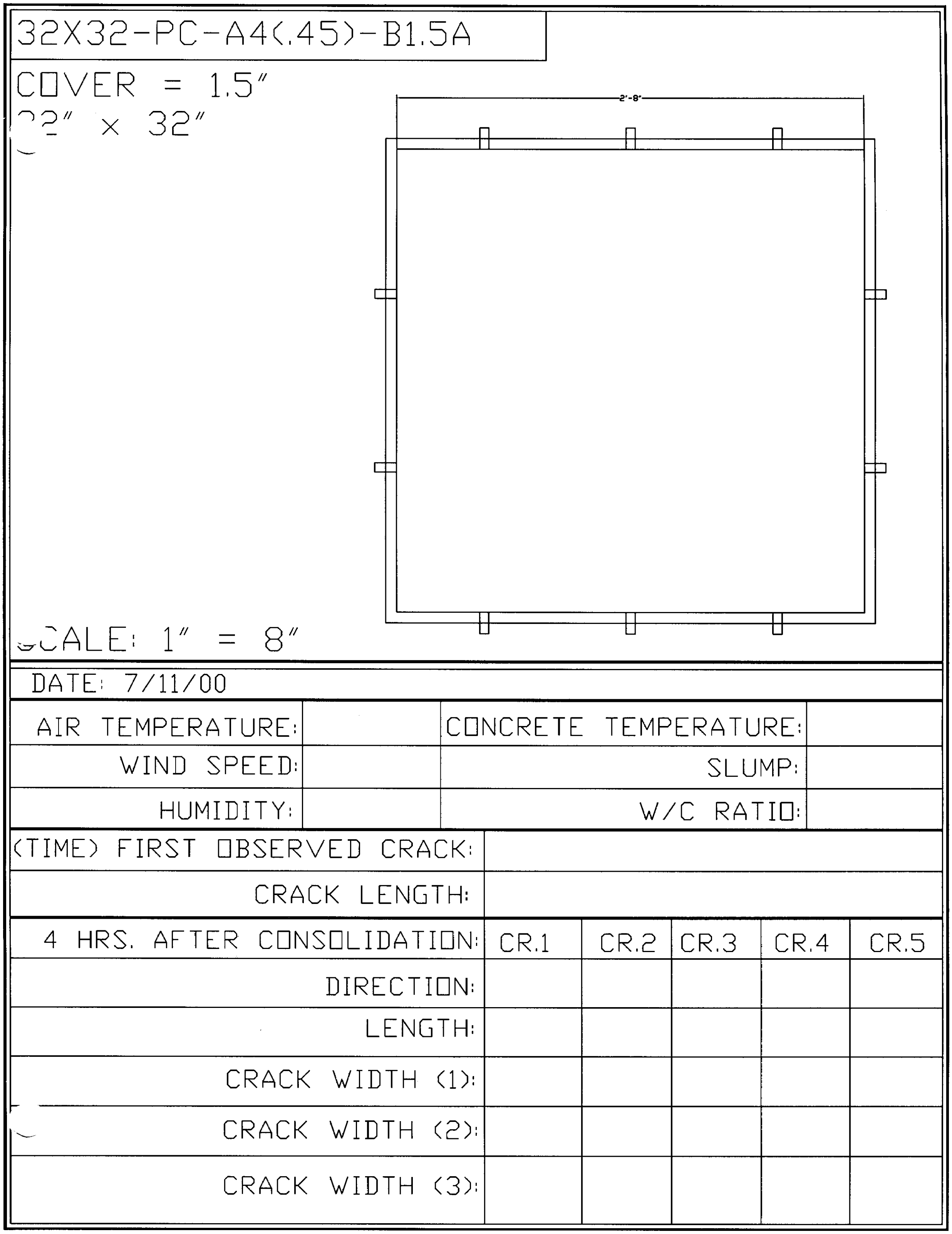
As previously stated, the concrete specimens were moist cured in their forms for three days and kept out of direct sunlight for an additional 15 days. Specimens were left in their forms and the top surface was allowed to dry outdoors under ambient conditions in Blacksburg, Virginia. Visual inspections were conducted again at one, three, six and eight months to identify additional cracking, crack width widening and or crack lengthening detected on the specimen top surfaces.

5.3.2 Measurement and Recording Technique

Crack measurements over the top reinforcing bars were of chief concern in this study. Additional measurements were taken when cracking was observed over the lower reinforcing bar.

For each crack width and length, the average of three measurements were recorded. Crack lengths were measured to the nearest 13 mm (0.5 in.), and crack widths to the nearest 0.02 mm (0.0008 in.). Cracks with lengths less than 13 mm (0.5 in.) in length were not recorded. Crack widths less than .05 mm (0.002 in.) were recorded as very small (VS).

Observations of the larger specimens (813 and 610 mm square specimens) were made systematically to provide uniform observation conditions. This was achieved by dividing the specimens into nine regions. Observation regions were defined by specimen boundary conditions (reinforcement bar intersections and form edges). Crack location, lengths and widths were recorded on individual specimen crack record diagrams. Various color pens were used for each measurement time to indicate at which time period the measurement was taken. Pencil was used to record initial observations, blue at one month, red at three months, black at six months and brown at eight months. Figure 5-4 is a typical sample crack record diagram used in this study. (Actual paper size: 200 x 280 mm (8.5 x 11 in.))



32X32-PC-A4(.45)-B1.5A

COVER = 1.5"
32" × 32"

2'-8"

SCALE: 1" = 8"

DATE: 7/11/00

AIR TEMPERATURE:		CONCRETE TEMPERATURE:	
WIND SPEED:		SLUMP:	
HUMIDITY:		W/C RATIO:	

(TIME) FIRST OBSERVED CRACK:	
CRACK LENGTH:	

4 HRS. AFTER CONSOLIDATION:	CR.1	CR.2	CR.3	CR.4	CR.5
DIRECTION:					
LENGTH:					
CRACK WIDTH (1):					
CRACK WIDTH (2):					
CRACK WIDTH (3):					

Figure 5-4: Sample Crack Record Diagram

5.4 Materials

5.4.1 Concrete Mixture

The two VDOT A4 bridge deck concrete mixtures were a PC mixture and a 60/40 percent PC/GGBFS blended cement concrete mixture. Ready mix truck batch number one was the Type I/II PC/GGBFS mixture and batches number two and three were the Type I/II PC mixtures. The coarse aggregate used was a VDOT size number 57, non-polishing crushed quartzite from Sylvatus, VA. The fine aggregate was a natural sand from Wytheville, VA. The concrete mixture contained a water reducer, had a maximum w/c ratio of 0.45, an air content of 7.5 ± 1.5%, and an allowable concrete slump range of 50 to 175 mm (2 to 7 in.). The concrete mixture proportions are presented in Appendix II.

5.4.2 Forms and Reinforcing Steel

The ECR and BRS were the same bar diameter (16 mm) and had the same deformation pattern. As indicated before, 19-mm (3/4 in.) plywood was used to fabricate the specimen forms. The plywood was A-C grade spruce-pine-fur using the A-graded side for the inside surfaces. The wood screws used for assembly were standard 41 mm (1-5/8 in.) coarse threaded phosphate-coated screws. The form oil was De-Lox, manufactured by W.R. Meadows, Inc.

6. RESULTS

6.1 Fresh and Hardened Concrete Properties

Table 6-1 presents the slump, concrete temperature, air content, unit weight and compressive strength for the three concrete truck batches. The mixtures were batched and delivered by the same concrete supplier on three separate days.

Table 6-1: Concrete Mixture Data as Delivered

Concrete Control Test	Number of Tests	Batch 1 PC/GGBFS	Batch 2 PC	Batch 3 PC
Initial Slump, mm (in)	1	89 (3.5)	83 (3.5)	152 (6)
Final Slump, mm (in)	1	76 (3)	57 (2.5)	127 (5)
Concrete Temperature,^{o}C (^{o}F)	3	29.2 (85)	28.9 (84)	27.5 (82)
Air Content, %	2	5	3.6	6
Unit Weight, kN/m^3 (lb/ft^3)	2	22.6 (143.5)	22.8 (145.1)	22.2 (140.9)
Compressive Strength, MPa (psi):				
3-Day	2	17.2 (2,500)	22.1 (3,200)	17.9 (2,600)
7-Day	2	24.8 (3,600)	24.8 (3,600)	21.4 (3,100)
28-Day	4	33.8 (4,900)	35.2 (5,100)	26.2 (3,800)
180-Days	2	41.4 (6,000)	41.4 (6,000)	29.6 (4,300)

The air contents ranged between 3.6 and 6.0 percent. Batch 1 and 2 had an insufficient air content of 5.0 and 3.6 percent; Table II-17 from the VDOT Road and Bridge Specifications require an air content of 7.5 ± 1.5 percent for class 30 (A4) concrete mixtures with high range water reducers [VDOT Road and Bridge Specifications, 1997]. Concrete slumps ranged between 83 and 152 mm which is within the requirements for VDOT A4 concrete bridge deck mixtures. VDOT specifications for water reduced Class 30 (A4) concrete require slump values within the range of 50 to 175 mm.

The average 28-day compressive strengths ranged between 26 and 35 MPa for the mixtures. Batch 3 (100% PC) had the highest air content, 6%, the greatest slump, 152 mm, and the lowest 28-day compressive strength, 26.2 MPa. VDOT specifications require a 28-day compressive strength of 27.6 MPa.

6.2 Ambient Weather Conditions During Specimen Casting

Average ambient temperatures ranged between 30.8 and 21.7 °C for the three placement days and the average relative humidity ranged between 62 and 82%. Wind was minimal during the concrete placement, consolidation and initial observation periods. Specimens cast from Batch 1 experienced the most amount of wind; gusts were measured having an average velocity of 3 mph.

Table 6-2: Ambient Weather Conditions

Environment Control	Number of Tests	Batch 1 PC/GGBFS	Batch 2 PC	Batch 3 PC
Ambient Temperature, ^{o}C (^{o}F)	3	30.8 (87.4)	25 (77.0)	21.7 (71.1)
Relative Humidity, (%)	4	62	82	79
Wind Speed, mph	5	3	0	0

6.3 Exposure Conditions

After three days moist curing, the specimens were subjected to normal ambient conditions for two weeks. The specimens were then moved to an on-site storage yard for continued observation. Summer weather conditions in Blacksburg Virginia were mild. The average high temperature for the area during the month of July was 25.3 °C (77.6 °F) and an average low of 15.2 °C (59.4 °F) [NOAA, 2000]. The month of July 2000 had a departure from historical average temperatures of – 1.15°C (– 2.1°F); this indicates that temperatures were mild. Weather reports for the area indicate that 104 mm (4.1 in.) of rain fell during the month of July.

6.4 Subsidence Cracking

6.4.1 Initial Observations

Few cracks were detected during the first six hours after concrete consolidation and finishing. The 813 mm (32 in.) square specimens, with a cover depth of 38 mm (1.5 in.), cast with Batch 1 (PC/GGBFS), exhibited cracking to a small degree between 4 and 5.5 hours. Six cracks were detected in 3 specimens directly above the top bars; 2 of the specimens contained ECR and 1 contained BRS. Crack lengths ranged between 127 and 76 mm (5 and 3 in). These cracks had widths smaller than measurable by a crack comparator. As a result, crack widths were approximated to be 0.05 mm.

Total crack lengths recorded were divided by the sum of the possible crack length, where the total possible crack length was taken as the sum of the lengths of the top reinforcing bars. This percent cracked value provided a measure of the degree of cracking experienced by each specimen. By representing the cracked specimens quantitatively as percent cracked, the effect of concrete cover depths, reinforcement bar spacing and type of concrete cement may be seen more clearly.

Tables 6-3 presents the cracking results for the initial observations; the number of specimens that cracked and the average specimen cracking percentage have been included. See Appendix I for more specific numerical data for each test specimen.

Table 6-3: Subsidence Cracking Observed During First Six Hours After Consolidation and Finishing.

Specimen Size	**813x813 mm (32x32 in.)**		**813x813 mm (32x32 in.)**		**610x610 mm (24x24 in.)**		**305x305 mm (12x12 in.)**	
	Batch 1		Batch 2		Batch 3		Batch 3	
Number of Specimens Cracked, (6 hours)	BRS	ECR	BRS	ECR	BRS	ECR	BRS	ECR
Concrete Cover, mm								
38.1 mm (1.5 in.)	1	2	0	0	0	0	0	0
44.5 mm (1.75 in.)	0	0	0	0	0	0	0	0
50.8 mm (2 in.)	0	0	0	0	0	0	0	0
57.2 mm (2.25 in.)					0	0	0	0
63.5 mm (2.5 in.)					0	0	0	0
Average Percent Cracked, (6 hours)	BRS	ECR	BRS	ECR	BRS	ECR	BRS	ECR
Concrete Cover, mm								
38.1 mm (1.5 in.)	3.47	5.03	0	0	0	0	0	0
44.5 mm (1.75 in.)	0	0	0	0	0	0	0	0
50.8 mm (2 in.)	0	0	0	0	0	0	0	0
57.2 mm (2.25 in.)							0	0
63.5 mm (2.5 in.)							0	0

6.4.2 Extended Study

Table 6-4 presents the results of crack observations made at 30 days of age. It may bee seen that the number of specimens cracked and percentage of cracking increased in each specimen as the concrete was exposed to normal ambient exposure conditions. As previously mentioned only three specimens had appeared to crack from the initial subsidence stresses. It was later observed that 19 specimens had cracked at 30 days (1 month) of age, 47 had cracked at 90 days (3 months), 61 had cracked at 180 days (6 months), and 69 had cracked at 240 days (8 months) of age. Table 6-5 through 6-7 presents the results of continued crack observation studies conducted at 90, 180, and 240 days. In addition, Figures 6-1 through 6-4 illustrate the crack propagation over time for each specimen size.

Table 6-4: Number of Cracked Specimens and Percent Cracking at 30 days.

Specimen Size	**813x813 mm (32x32 in.)**		**813x813 mm (32x32 in.)**		**610x610 mm (24x24 in.)**		**305x305 mm (12x12 in.)**	
	Batch 1		Batch 2		Batch 3		Batch 3	
Number of Specimens Cracked, (1 Month)	BRS	ECR	BRS	ECR	BRS	ECR	BRS	ECR
Concrete Cover, mm								
38.1 mm (1.5 in.)	3	3	2	3	0	0	2	2
44.5 mm (1.75 in.)	1	2	0	0	0	0	0	1
50.8 mm (2 in.)	0	0	0	0	0	0	0	0
57.2 mm (2.25 in.)							0	0
63.5 mm (2.5 in.)							0	0
Average Percent Cracked, (1 Month)	BRS	ECR	BRS	ECR	BRS	ECR	BRS	ECR
Concrete Cover, mm								
38.1 mm (1.5 in.)	22.7	23.3	2.3	5.9	0	0	18.1	29.2
44.5 mm (1.75 in.)	0.66	1.66	0	0	0	0	0	2.78
50.8 mm (2 in.)	0	0	0	0	0	0	0	0
57.2 mm (2.25 in.)							0	0
63.5 mm (2.5 in.)							0	0

Table 6-5: Number of Cracked Specimens and Percent Cracking at 90 days.

Specimen Size	**813x813 mm (32x32 in.)**		**813x813 mm (32x32 in.)**		**610x610 mm (24x24 in.)**		**305x305 mm (12x12 in.)**	
	Batch 1		Batch 2		Batch 3		Batch 3	
Number of Specimens Cracked, (3 Month)	BRS	ECR	BRS	ECR	BRS	ECR	BRS	ECR
Concrete Cover, mm								
38.1 mm (1.5 in.)	3	3	3	3	3	3	3	3
44.5 mm (1.75 in.)	3	3	0	0	3	2	3	2
50.8 mm (2 in.)	3	3	0	0	0	0	1	0
57.2 mm (2.25 in.)							0	0
63.5 mm (2.5 in.)							0	0
Average Percent Cracked, (3 Month)	BRS	ECR	BRS	ECR	BRS	ECR	BRS	ECR
Concrete Cover, mm								
38.1 mm (1.5 in.)	51.0	40.3	2.6	5.9	7.2	8.8	37.5	43.1
44.5 mm (1.75 in.)	20.3	29.0	0.0	0.0	6.9	6.3	23.6	27.8
50.8 mm (2 in.)	9.2	16.0	0.0	0.0	0.0	0.0	2.8	0.0
57.2 mm (2.25 in.)							0	0
63.5 mm (2.5 in.)							0	0

Table 6-6: Number of Cracked Specimens and Percent Cracking at 180 days.

Specimen Size	**813x813 mm (32x32 in.)**		**813x813 mm (32x32 in.)**		**610x610 mm (24x24 in.)**		**305x305 mm (12x12 in.)**	
	Batch 1		Batch 2		Batch 3		Batch 3	
Number of Specimens Cracked, (6 Month)	BRS	ECR	BRS	ECR	BRS	ECR	BRS	ECR
Concrete Cover, mm								
38.1 mm (1.5 in.)	3	3	3	3	3	3	3	3
44.5 mm (1.75 in.)	3	3	3	3	3	3	3	2
50.8 mm (2 in.)	3	3	1	1	1	1	1	1
57.2 mm (2.25 in.)							2	0
63.5 mm (2.5 in.)							0	0
Average Percent Cracked, (6 Month)	BRS	ECR	BRS	ECR	BRS	ECR	BRS	ECR
Concrete Cover, mm								
38.1 mm (1.5 in.)	60.1	64.4	13.2	14.2	17.8	15.2	59.0	56.9
44.5 mm (1.75 in.)	30.0	41.4	11.6	5.2	11.6	14.8	27.8	29.2
50.8 mm (2 in.)	20.1	23.6	2.1	1.6	2.8	2.1	2.8	4.2
57.2 mm (2.25 in.)							13.9	0
63.5 mm (2.5 in.)							0	0

Table 6-7: Number of Cracked Specimens and Percent Cracking at 240 days.

Specimen Size	**813x813 mm (32x32 in.)**		**813x813 mm (32x32 in.)**		**610x610 mm (24x24 in.)**		**305x305 mm (12x12 in.)**	
	Batch 1		Batch 2		Batch 3		Batch 3	
Number of Specimens Cracked, (8 Month)	BRS	ECR	BRS	ECR	BRS	ECR	BRS	ECR
Concrete Cover, mm								
38.1 mm (1.5 in.)	3	3	3	3	3	3	3	3
44.5 mm (1.75 in.)	3	3	3	3	3	3	3	2
50.8 mm (2 in.)	3	3	3	2	3	2	1	1
57.2 mm (2.25 in.)							2	2
63.5 mm (2.5 in.)							0	0
Average Percent Cracked, (8 Month)	BRS	ECR	BRS	ECR	BRS	ECR	BRS	ECR
Concrete Cover, mm								
38.1 mm (1.5 in.)	60.9	64.8	13.5	14.2	19.7	17.9	67.4	56.9
44.5 mm (1.75 in.)	30.9	42.1	13.4	7.1	15.4	16.2	31.9	29.2
50.8 mm (2 in.)	20.5	24.0	3.6	2.6	3.0	8.3	4.2	5.6
57.2 mm (2.25 in.)							13.9	2.08
63.5 mm (2.5 in.)							0	0

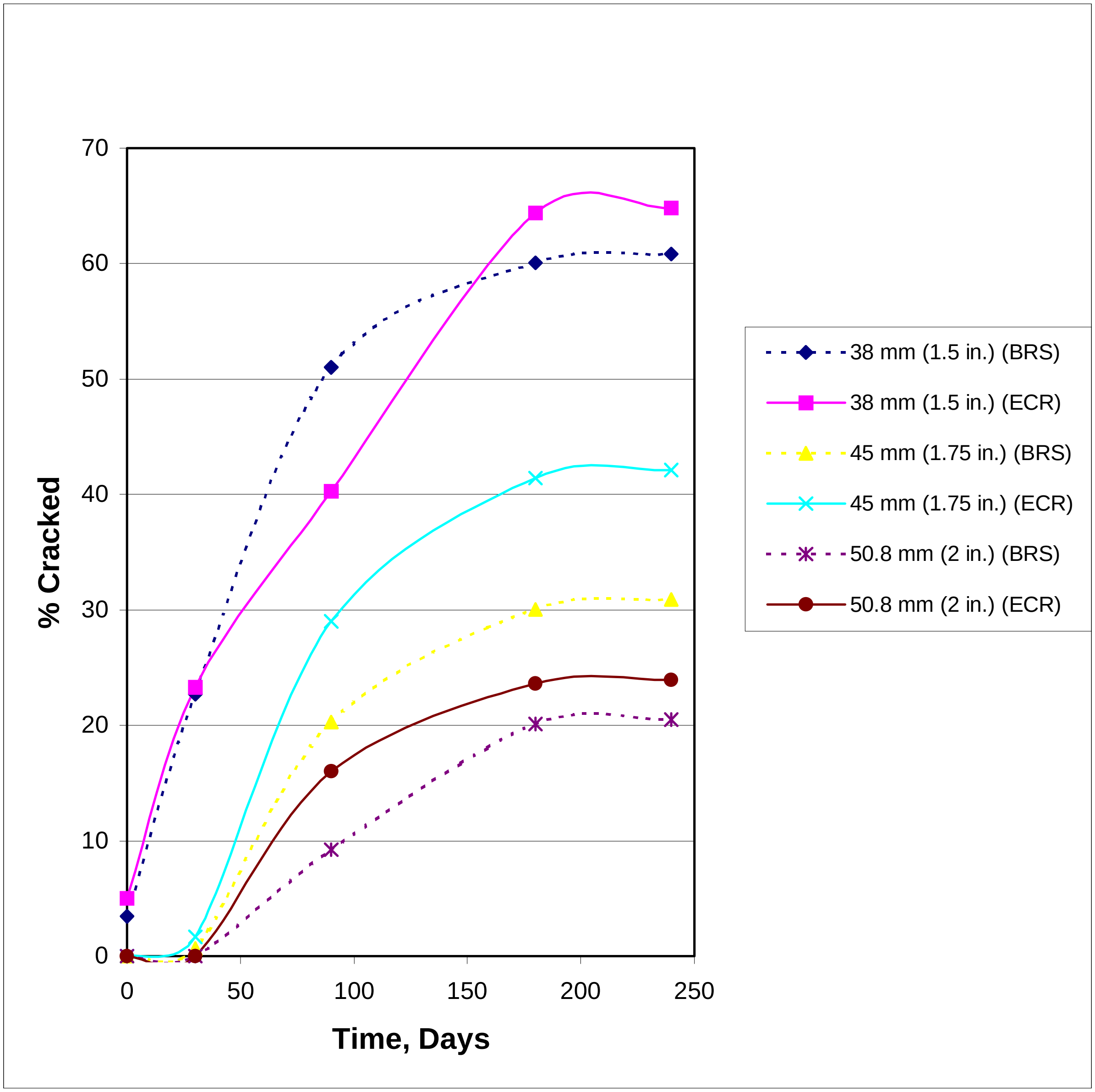


Figure 6-1: Percent Surface Cracking Propagation in 813 mm (32 in.) Square Specimens, GGBFS/PC (Batch 1).

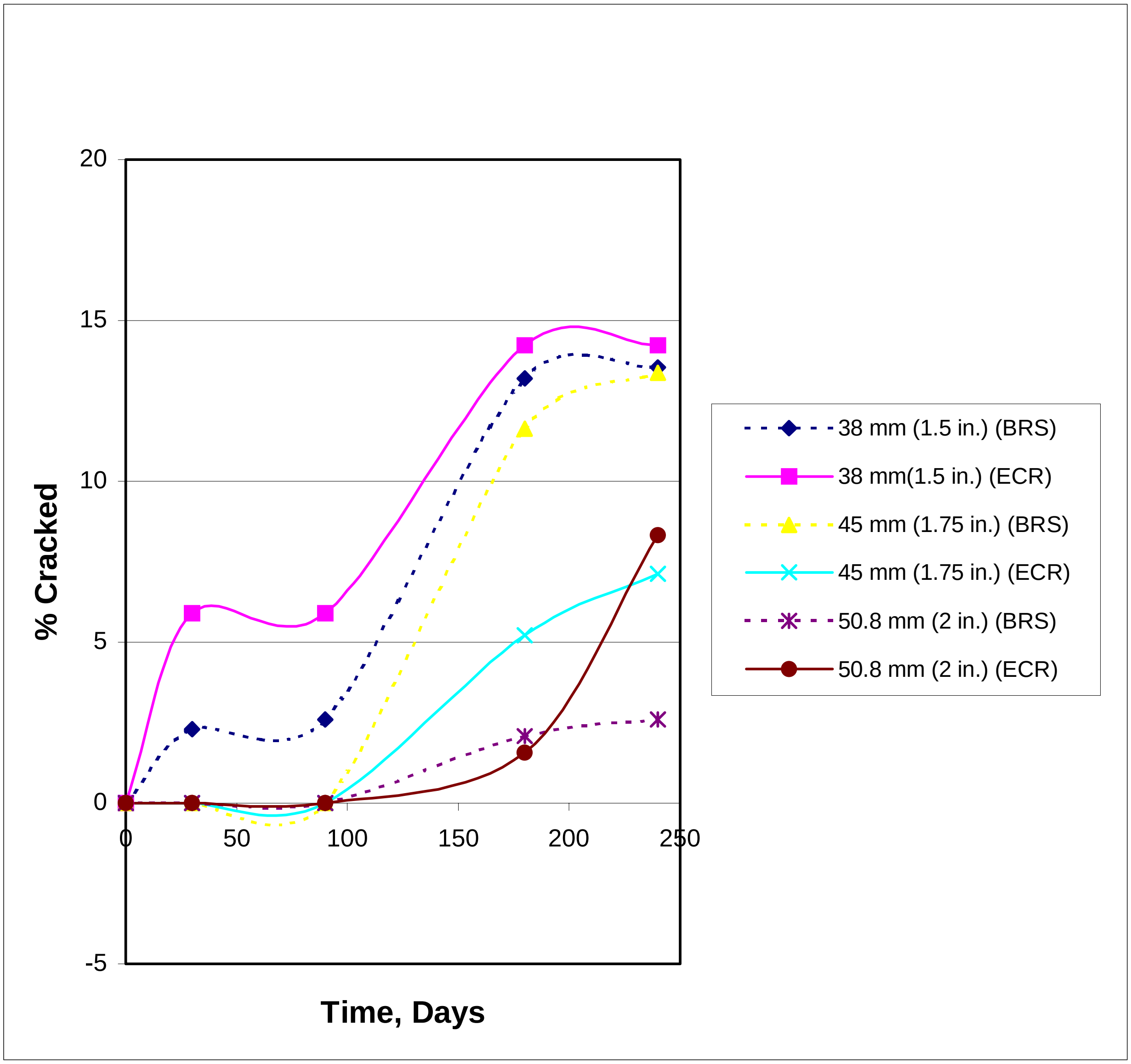


Figure 6-2: Percent Surface Cracking Propagation in 813 mm (32 in.) Square Specimens, PC (Batch 2).

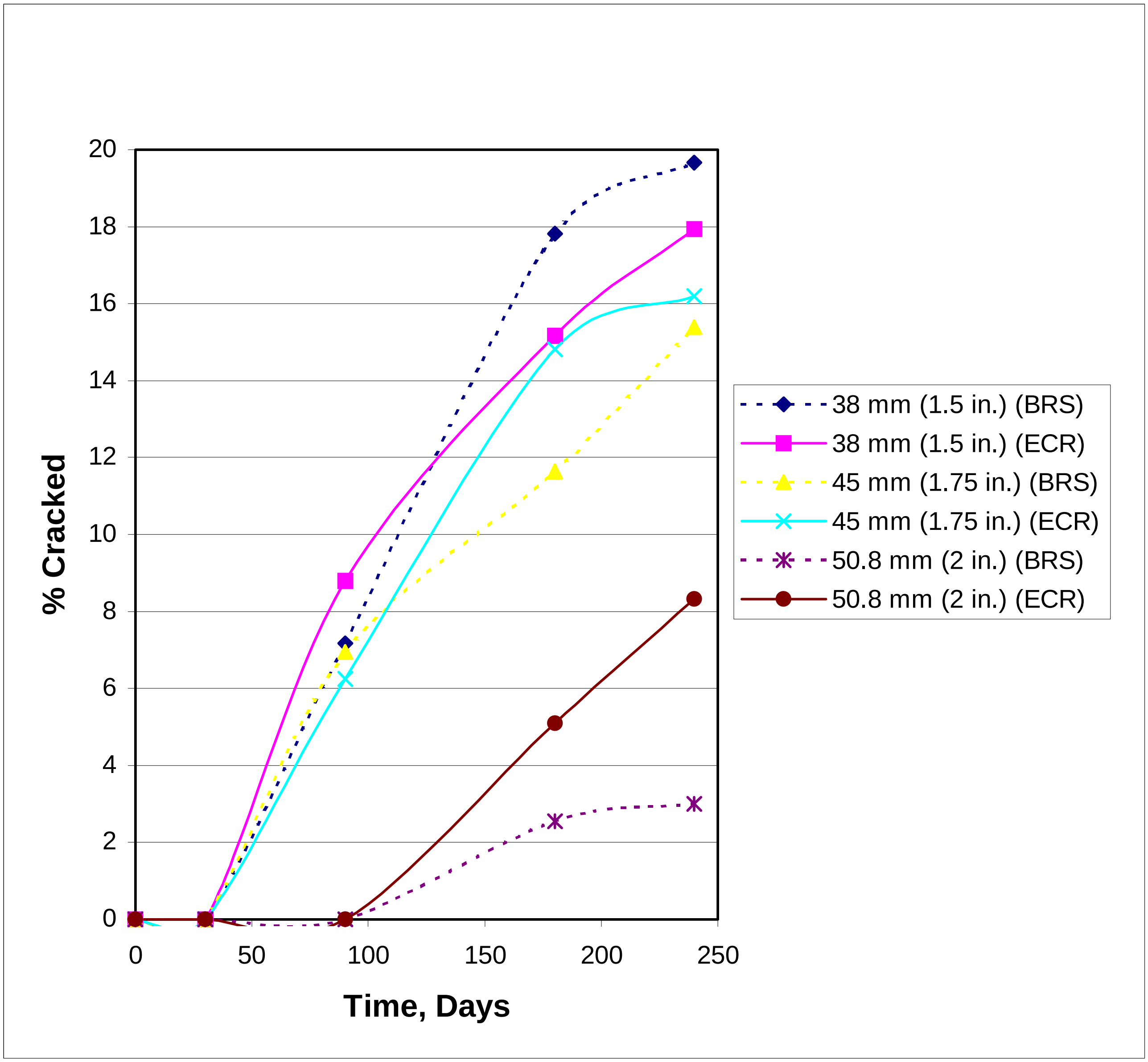


Figure 6-3: Percent Surface Cracking Propagation in 610 mm (24 in.) Square Specimens, PC (Batch 3).

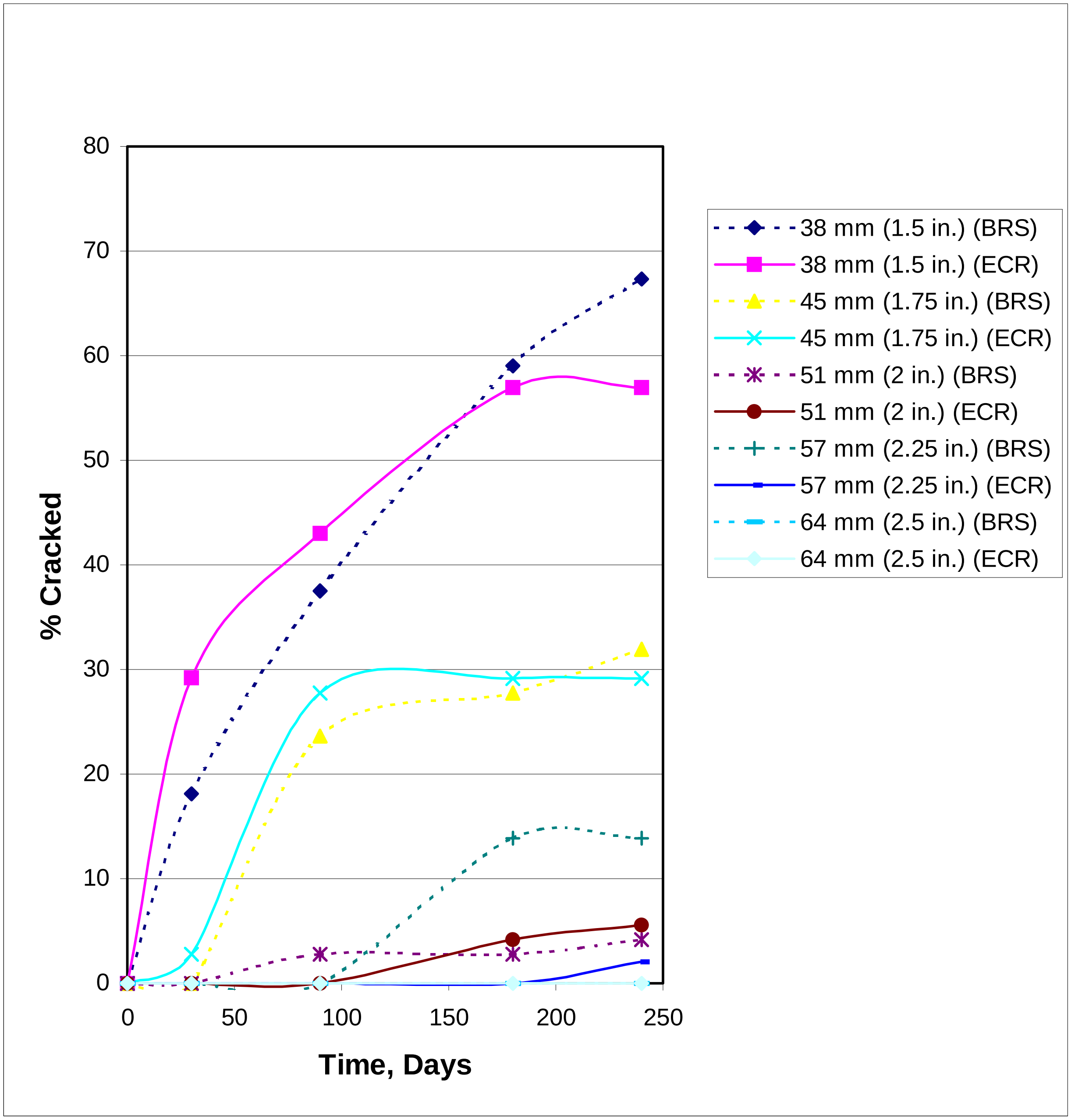


Figure 6-4: Percent Surface Cracking Propagation in 305 mm (12 in.) Square Specimens, PC (Batch 3).

The effect that ECR may have on the probability of subsidence cracking was a primary concern in this study. Almost every sample group at some point during the test showed that specimens containing ECR exhibited a higher degree of cracking as compared to BRS samples of the same size and cover depth. In general, specimens cast with ECR had cracking levels 0 to ± 10 percent that of the BRS specimens; see Figure 6-1 through 6-4.

In addition to the affect of ECR on subsidence cracking, concrete mixture type and reinforcing bar spacing factor were considered. As previously mentioned, two concrete mixtures were studied, a PC/GGBFS blended cement concrete (Batch 1) and a PC concrete (Batches 2 and 3). Two bar spacings and three form sizes were used to show the degree to which concrete boundary conditions affect subsidence cracking. Figure 6-5 through 6-7 present the results of observed cracking at six months of age relative to concrete mixture type and boundary conditions for three cover depths. The error bars indicate the 95% confidence interval for the crack measurements recorded. It is important to be aware that the specimens were cast from three separate concrete batches and limited comparisons between batches can be made.

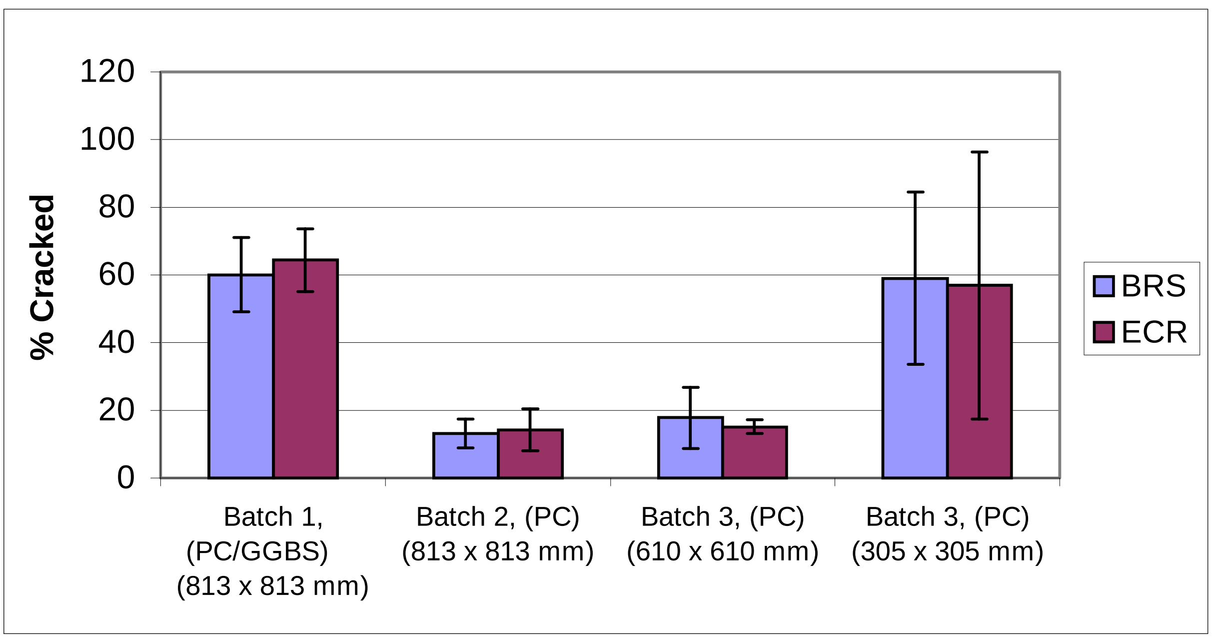


Figure 6-5: Percent Cracking in Specimens, 38 mm (1.5 in.) Concrete Cover Depth, 180 Days.

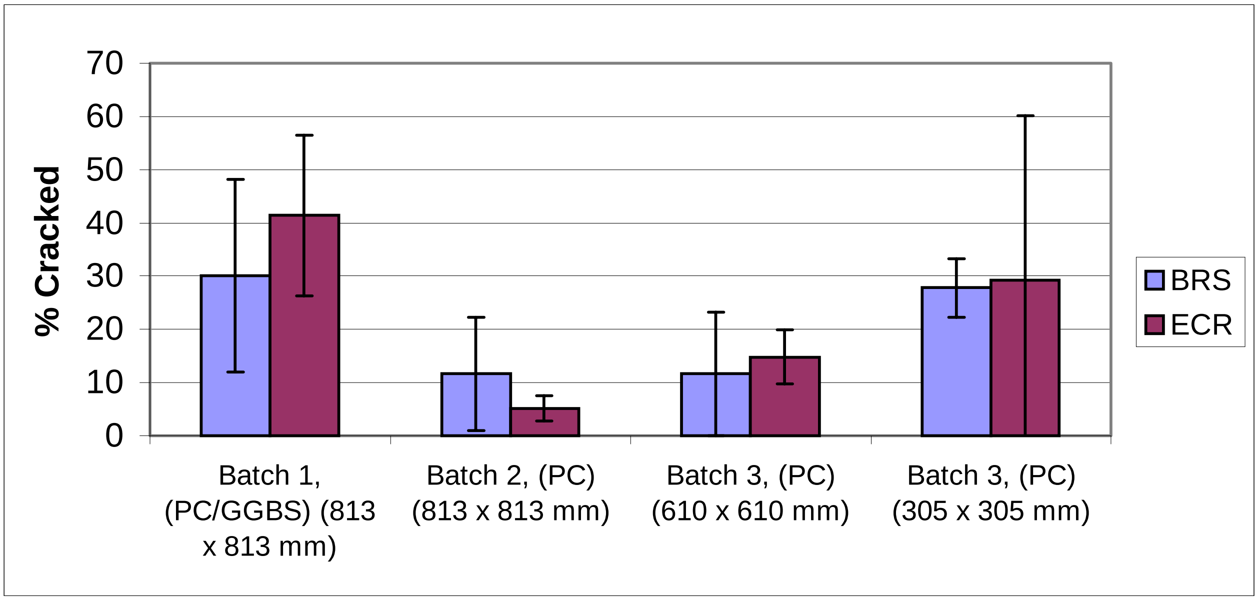


Figure 6-6: Percent Cracking in Specimens, 44.5 mm (1.75 in.) Concrete Cover Depth, 180 Days.

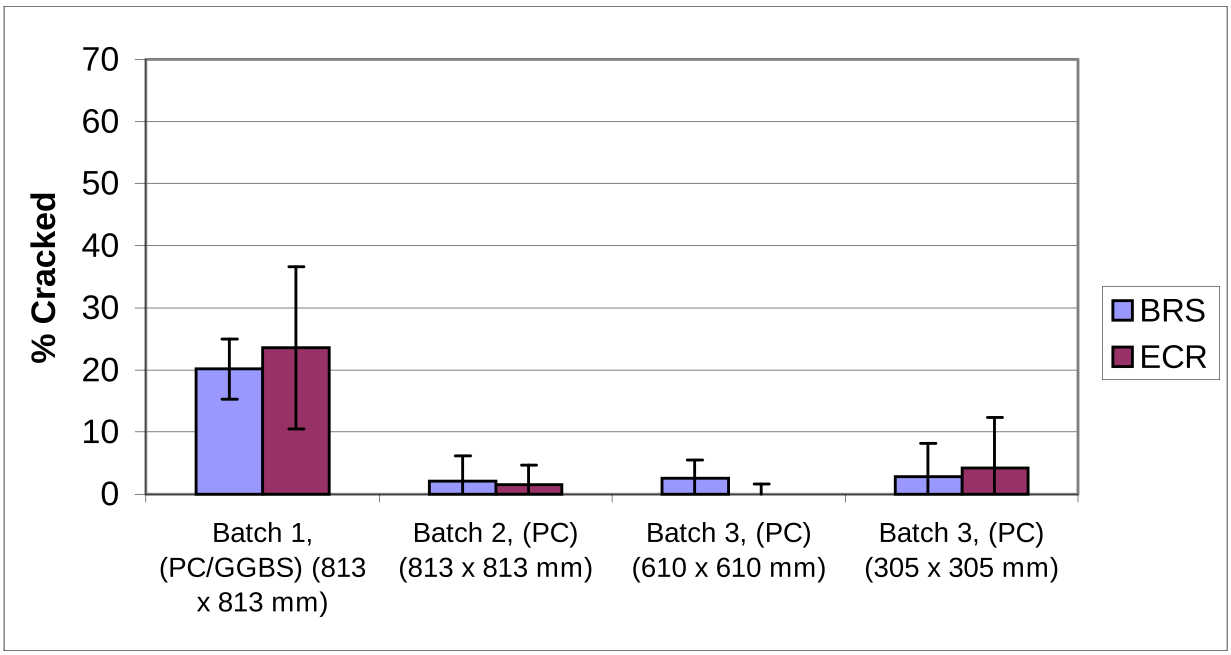


Figure 6-7: Percent Cracking in Specimens, 50.8 mm (2.0 in.) Concrete Cover Depth, 180 Days.

Variations in specimen cracking may be seen relative to the concrete mixture type used in this study. Figure 6-5 through 6-7 show that the PC/GGBFS concrete mixture at six months exhibited higher levels of cracking than the other two PC mixtures for all cover depths compared. It may be seen that the 813 mm (32 in.) square PC/GGBFS specimens having a 38.1 mm (1.5 in.) cover depth experienced an average cracking level of 62 percent whereas same size PC specimens were only 14 percent cracked.

Other variations in cracking may be seen by making a comparison of cracking levels relative to the specimen boundary conditions (form size). This may be observed by comparing the cracking results for the various specimen sizes cast in this study. Figure 6-5 through 6-7 show that specimen cracking at six months of age was more prevalent in the 305 mm (12 in.) square specimens than the 610 mm (24 in.) square specimens. It may be seen that the 305 mm (12 in.) square specimens had an average subsidence cracking level of 58 percent whereas the 610 mm (24 in.) square specimens were only 16 percent cracked. This is of particular interest to this study since these specimens were cast from the same batch (Batch 3).

Variation due to reinforcement bar spacing were however not as apparent. This may be seen by making a comparison of cracking observed between the 610 mm (Batch 3) and 813 mm (Batch 2) specimen sizes. Little variation in cracking was detected for both the ECR and BRS specimens regardless of the clear cover present. The greatest difference in cracking, 10%, was seen in the specimens cast with ECR, having a clear cover of 44 mm (1.75 in.). It should be made clear that the 610 and 813 mm square specimens were cast with the same concrete mixture design in two separate batches. Therefore, comparisons between these specimen types are limited due to concrete batch variations.

It was seen throughout this study that the greatest crack width for a specimen increased slightly as a function of total specimen cracking percent. Specimens cast with the PC/GGBFS showed the greatest increases in crack width. Crack widths were as large as 0.1 mm for specimens cracked greater than 50 percent; specimens cracked less than 50% had widths less than or equal to 0.05 mm. Figure 6-8 shows the relationship observed in the PC/GGBFS (Batch 1) specimens. Crack widths were very small in the two PC concrete mixtures; no measurable increases in crack widths could be observed. This may be seen in Figure 6-9, 6-10 and 6-11.

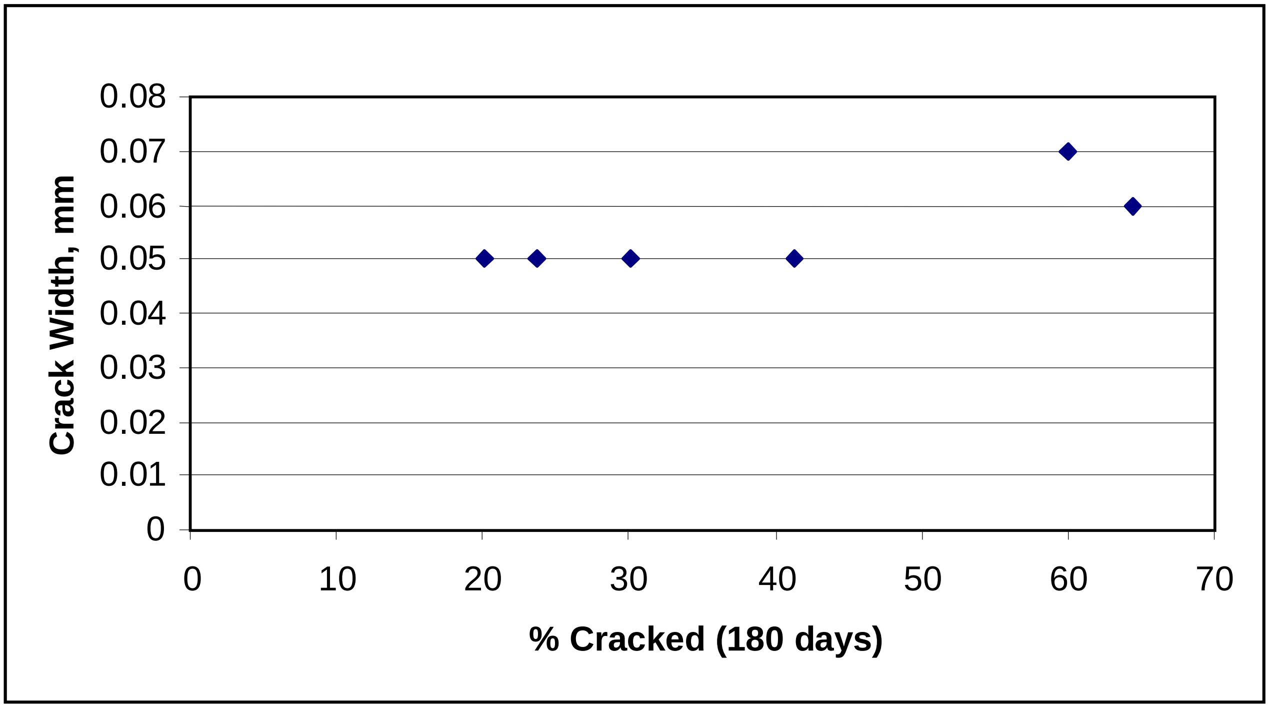


Figure 6-8: Surface Crack Width as a Function of Percent Surface Cracking, 813 mm (32 in.) Square, PC/GGBFS Concrete (Batch 1).

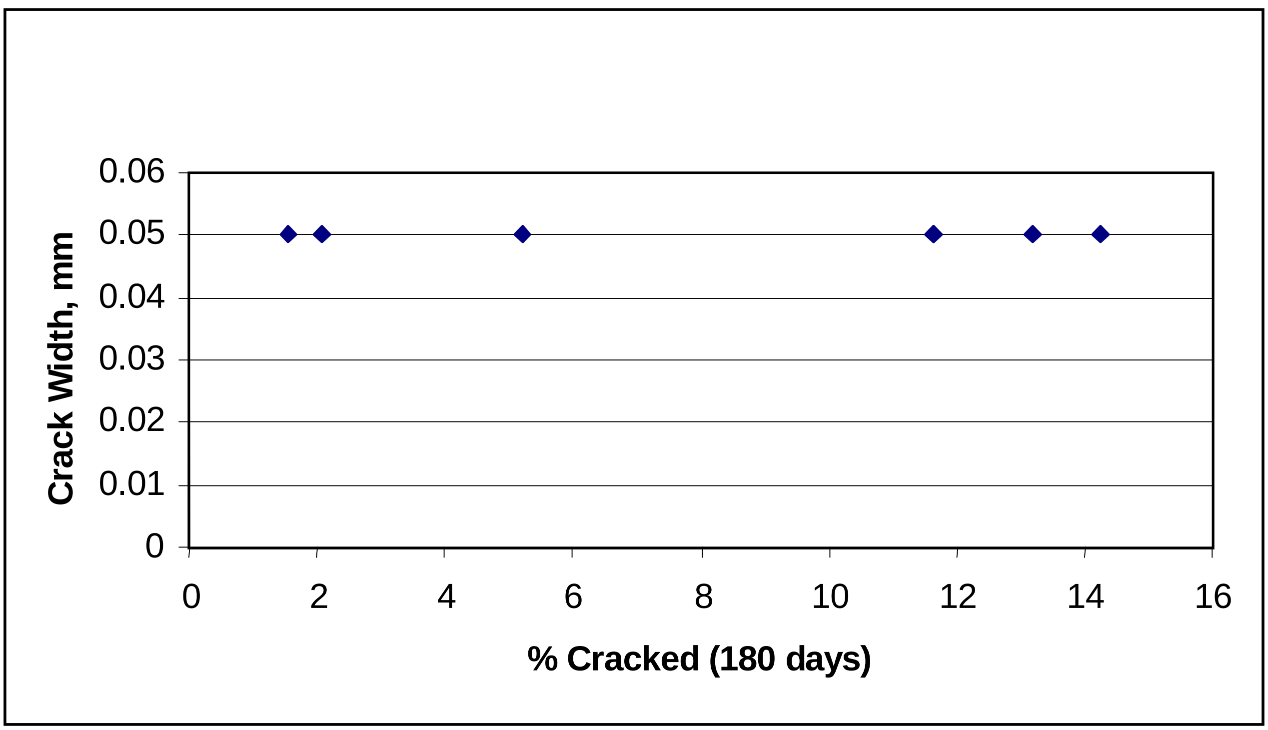


Figure 6-9: Surface Crack Width as a Function of Percent Surface Cracking, 813 mm (32 in.) Square, PC Concrete (Batch 2).

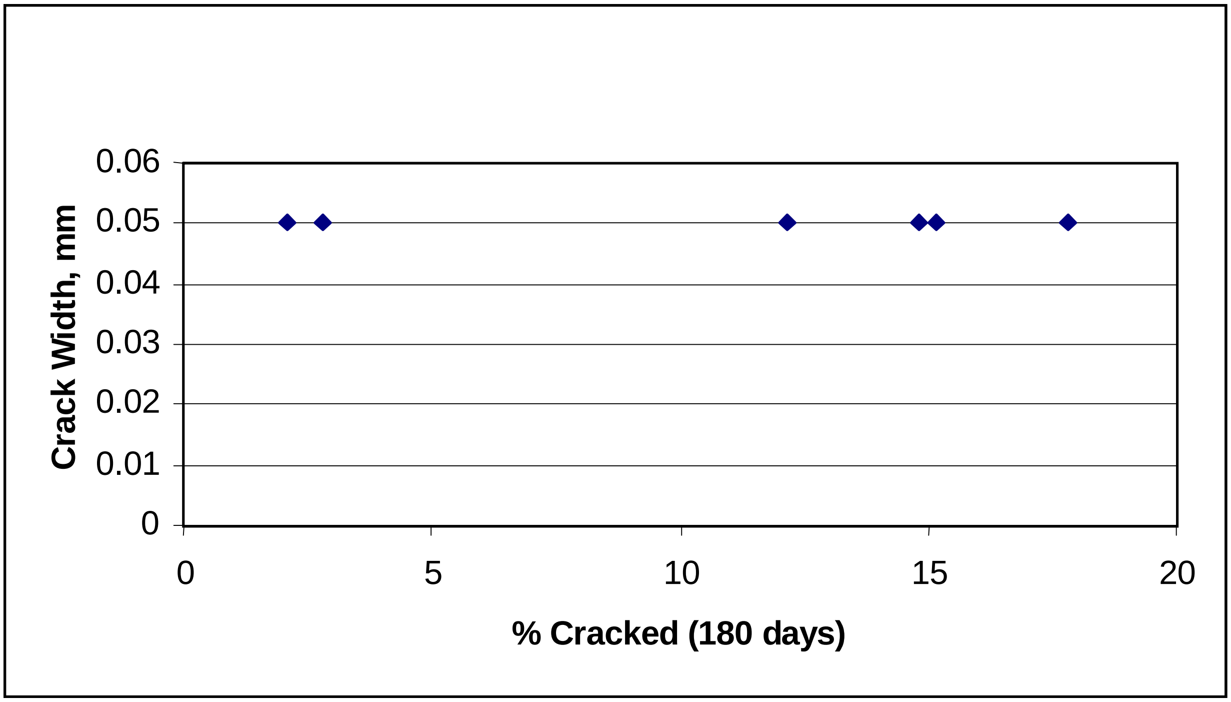


Figure 6-10: Surface Crack Width as a Function of Percent Surface Cracking, 610 mm (24 in.) Square, PC Concrete (Batch 3).

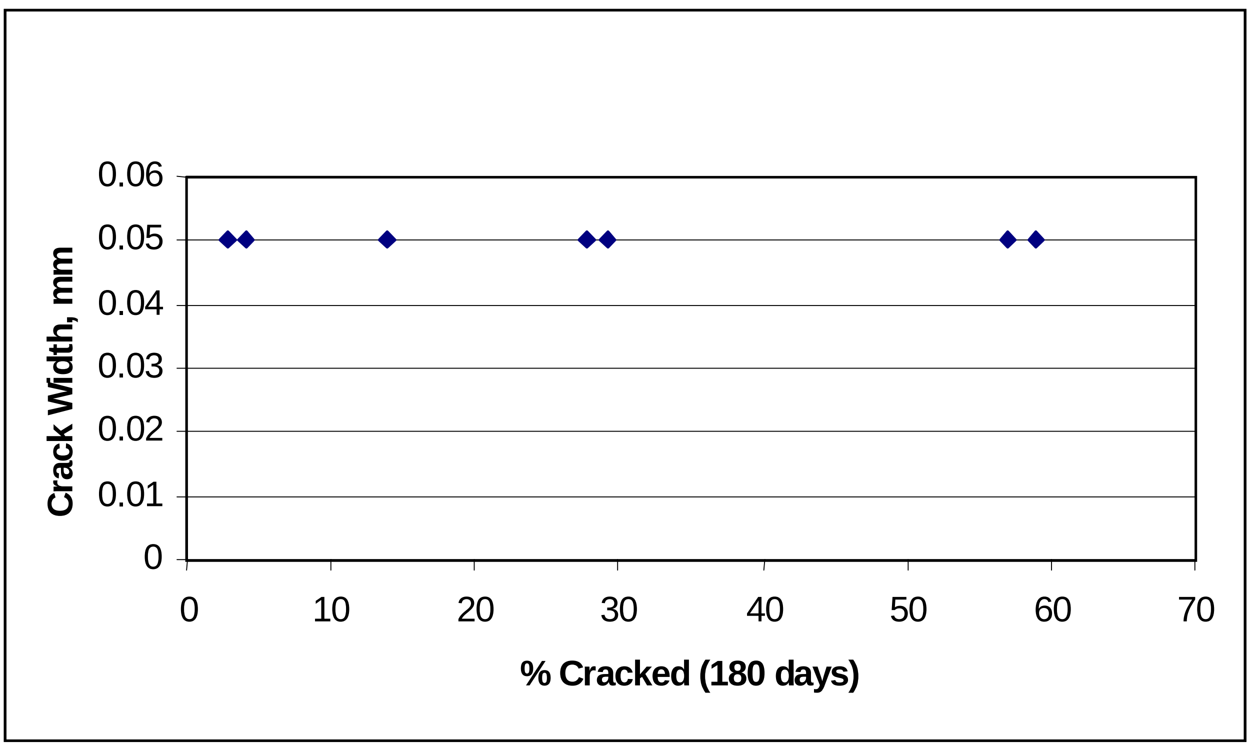


Figure 6-11: Surface Crack Width as a Function of Percent Surface Cracking, 305 mm (12 in.) Square, PC Concrete (Batch 3).

7. ANALYSIS AND DISCUSSION

7.1 Introduction

The objective of this study was to determine the effect that epoxy coated reinforcement may have on the occurrence of subsidence cracking over reinforcing bars. Other factors were bar spacing, single and multiple bars (boundary conditions), concrete mixture type and clear cover depth. Testing was conducted over a range of concrete clear cover depths for each study variable. To examine the experimental results from observations made at 6 months, this analysis used a number of multiple linear regression models to determine the relationship that reinforcing bar type, bar spacing, and concrete mixture type may have on the degree (percent) of subsidence cracking over a range of cover depths.

7.2 Statistical Analysis

A statistical modeling program, "MINITAB", was used to formulate linear regression models. For each set of data, MINITAB iterated combinations of the independent variables through various linear and curvilinear functions to determine the best-fit curve. For each of the models developed, the best fit regression curve was found to be linear, having the form:

$$Y = \beta_o + \beta_1(x_1) + \beta_2(x_2) + \ ..... + \beta_i(x_i)$$

where,

Y = Dependent variable

β_o = Coefficient relating to the conditions experienced when all variables are set to zero (y-intercept).

β_i = Coefficient relating to the impact of the independent variable (x_i).

x_i = Indicator variable that defines the test condition and is set at a 0 or 1 for every variable but concrete cover depth. The variable will be a 1 if the condition is present, otherwise the value will be zero.

7.2.1 Preliminary Statistical Models

Six preliminary multiple linear regression models were identified. Table 7-1 presents the predictor variables for the six models identified.

Table 7-1: Multiple Linear Regression Models Studied

Model Number	Source Data			Model Predictor Variables
	Specimen Size	Bar Spacing	Batch Number	
1	305 mm (12 in.)	6 in.	Batch 3	Bar Type, Clear Cover
2	305 mm (12 in.) 610 mm (24 in.)	6 in.	Batch 3	Bar Type, Clear Cover, Specimen Size
3	610 mm (24 in.)	6 in.	Batch 3	Bar Type, Clear Cover
4	805 mm (32 in.)	8 in.	Batch 2	Bar Type, Clear Cover
5	805 mm (32 in.)	8 in.	Batch 1	Bar Type, Clear Cover
6	305 mm (12 in.) 610 mm (24 in.) 805 mm (32 in.)	6 in. 8 in.	Batch 1 Batch 2 Batch 3	Bar Type, Clear Cover, Specimen Size, Bar Spacing, Batch Number

- Model 1 was constructed from the 305 mm (12 in.) size specimens cast from concrete Batch 3 (PC). This model contains the following variables: reinforcing bar type, and concrete clear cover where D represents the degree (%) of surface cracking experienced.

  Cracking Model 1:

  D = 131 –55.0(COVER) – 2.63(ECR)

- Model 2 was constructed from the 610 mm (32 in.) size specimens cast from concrete Batch 3 (PC). This model contains the following variables: bar type and concrete clear cover.

  Cracking Model 3:

  D = 54.7 – 25.3(COVER) – 1.32(ECR)

- Model 3 was constructed from the 813 mm (32 in.) size specimens cast from concrete Batch 2 (PC). This model contains the following variables: bar type and concrete clear cover.

  Cracking Model 4:

  D = 50.6 – 23.8(COVER) – 1.98(ECR)

- Model 4 was constructed from the 813 mm (32 in.) size specimens cast from concrete Batch 1 (GGBFS/PC). This model contains the following variables: bar type and concrete clear cover.

  Cracking Model 5:

  D = 178 – 80.7(COVER) + 6.40(ECR)

- Model 5 was constructed from the 305 mm (12 in.) and 610 mm (24 in.) size specimens cast from concrete Batch 3 (PC). This model contains the following variables: bar type, concrete clear cover and specimen size.

  Cracking Model 5:

  D = 120 – 50.1(COVER) – 1.15(ECR) – 20.8(PC_610)

- Model 6 was constructed from all the specimens in the study (305, 610 and 813 mm specimen sizes). This model contains the following variables: bar type, concrete clear cover, bar spacing factor and concrete mixture type.

  Cracking Model 6:

  D = 128 – 7.92(PC_305) – 28.9(PC_610) – 32.0(PC_813) – 50.6(COVER) + 0.29(ECR)

Where model parameters are:

D = Degree of surface cracking experienced, %.

GGBFS_813 = Indicator variable that identifies the influence of the ground granulated blast furnace slag cement (GGBFS, Batch 1), 813 mm (32 in.) square specimens.

PC_813 = Indicator variable that identifies the influence of the portland cement (PC, Batch 2), 813 mm (32 in.) square specimens.

PC_610 = Indicator variable that identifies the influence of the PC (Batch 3), 610 mm (24 in.) square specimens.
PC_305 = Indicator variable that identifies the influence of the PC (Batch 3), 305 mm (12 in.) square specimens.

ECR = Indicator variable that identifies the influence of reinforcing bar type (epoxy coated reinforcing steel).

COVER = Concrete clear cover, in.

7.2.1.1 Residual Analysis

Residual analyses demonstrated that the models did not exhibit quadratic or curvilinear properties but were multiple variable linear relationships.

7.2.1.2 Statistical F Test

The F-test was used to test the null hypothesis of H_o: β_1,β_2, β_3, β_4, $\beta_5 = 0$ at a level of significance of 5%. The results of the F test have been presented in Table 7-2.

Table 7-2: Results of Statistical F-test at a 95% Level of Significance.

Model Number	F (Model)	Fu (k,n-k-1)	Accept Hypothesis if F < Fu
1	17.03	3.35	Reject
2	7.89	3.68	Reject
3	9.06	3.68	Reject
4	22.21	3.68	Reject
5	17.54	2.84	Reject
6	30.85	2.35	Reject

The calculated F values for each regression model, which were greater than the tabulated Fu value at a significance level of 5%, rejected the null hypothesis, H_o: $\beta_1,\beta_2, \beta_3, \beta_4, \beta_5 = 0$. Thus, some of the explanatory variables were found to be significant or greater than zero.

7.2.1.3 Statistical t Test

The t test was used to determine if the independent variables individually had a significant influence on the degree of cracking. A null hypothesis of H_o: $\beta_i = 0$ was tested for each variable at a 5% significance level. The degree of freedom for the sample was defined as the number of specimens, n, subtracted by one more than the number of explanatory variables, k, in the model (n – k – 1). The tabulated t values for the number of degrees of freedom were compared with the calculated t test values. The t test variables were calculated by dividing the coefficient, β, by its estimated standard deviation. The t test results for each of the six models identified are presented in Tables 7-3a through 7-3f. Each of the predictor variables were significant at the 5% confidence level except for the ECR predictor variable. Thus, there appears to be no significant difference in surface cracking between bare bar and ECR specimens.

Table 7-3a: Results of Statistical t Test for Model 1 (Batch 3, 305 mm).

Predictor Variable	Coefficient	Standard Deviation, s	t Test Value	Accept Hypothesis if | t | < 2.052
Constant	130.71	19.48	6.71	Reject
COVER	-55.007	9.447	-5.82	Reject
ECR	-2.633	6.68	-0.39	Fails to Reject

Table 7-3b: Results of Statistical t Test for Model 2 (Batch 3, 610 mm).

Predictor Variable	Coefficient	Standard Deviation, s	t Test Value	Accept Hypothesis if | t | < 2.132
Constant	54.66	11.39	4.80	Reject
COVER	-25.3	6.421	-3.94	Reject
ECR	1.32	2.621	0.50	Fails to Reject

Table 7-3c: Results of Statistical t Test for Model 3 (Batch 2, 813 mm).

Predictor Variable	Coefficient	Standard Deviation, s	t Test Value	Accept Hypothesis if | t | < 2.132
Constant	50.57	10.11	5.00	Reject
COVER	-23.767	5.698	-4.17	Reject
ECR	-1.978	2.326	-0.85	Fails to Reject

Table 7-3d: Results of Statistical t Test for Model 4 (Batch 1, 813 mm).

Predictor Variable	Coefficient	Standard Deviation, s	t Test Value	Accept Hypothesis if | t | < 2.132
Constant	177.97	21.88	8.13	Reject
COVER	-80.70	12.33	-6.54	Reject
ECR	6.40	5.036	1.27	Fails to Reject

Table 7-3e: Results of Statistical t Test for Model 5 (Batch 3, 305 and 610 mm).

Predictor Variable	Coefficient	Standard Deviation, s	t Test Value	Accept Hypothesis if | t | < 2.037
Constant	120.06	14.71	8.16	Reject
COVER	-50.056	7.141	-7.01	Reject
ECR	-1.15	4.373	-0.26	Fails to Reject
PC_610	20.841	4.856	-4.29	Reject

Table 7-3f: Results of Statistical t Test for Model 6 (All Specimens).

Predictor Variable	Coefficient	Standard Deviation, s	t Test Value	Accept Hypothesis if | t | < 1.991
Constant	129.13	10.03	12.88	Reject
PC_305	-7.83	4.16	-1.88	Fails to Reject
PC_610	-29.13	4.40	-6.62	Reject
PC_813	31.96	4.40	-7.27	Reject
COVER	-50.95	5.39	-9.46	Reject
ECR	-0.04	2.88	-0.01	Fails to Reject

7.2.1.4 Statistical p Value Comparison

The observed level of significance or p-value may also indicate if a variable in a multiple linear regression model is significant. The p-value is the probability of obtaining a test result more contradictory to the null hypothesis of a zero coefficient than the result obtained from the sample data given that the null hypothesis H_o is true. If the computed p value is greater than or equal to the level of significance (0.05) the null hypothesis is not rejected and the variable is insignificant. Tables 7-4a through 7-4f present the p value test results. Every variable but the reinforcing bar type (ECR) was found to be significant in each model.

Table 7-4a: Statistical p Value Approach for Model 1 (Batch 3, 305 mm).

Predictor Variable	Coefficient	p-Value	Accept Hypothesis if | p | > 0.05
Constant	130.71	0.000	Reject
COVER	-55.007	0.000	Reject
ECR	-2.633	0.39	Fails to Reject

Table 7-4b: Statistical p Value Approach for Model 2 (Batch 3, 610 mm).

Predictor Variable	Coefficient	p-Value	Accept Hypothesis if | p | > 0.05
Constant	54.66	0.000	Reject
COVER	-25.3	0.001	Reject
ECR	1.32	0.621	Fails to Reject

Table 7-4c: Statistical p Value Approach for Model 3 (Batch 2, 813 mm).

Predictor Variable	Coefficient	p-Value	Accept Hypothesis if | p | > 0.05
Constant	50.57	0.000	Reject
COVER	-23.767	0.001	Reject
ECR	-1.978	0.409	Fails to Reject

Table 7-4d: Statistical p Value Approach for Model 4 (Batch 1, 813 mm).

Predictor Variable	Coefficient	p-Value	Accept Hypothesis if | p | > 0.05
Constant	177.97	0.000	Reject
COVER	-80.70	0.000	Reject
ECR	6.40	0.223	Fails to Reject

Table 7-4e: Statistical p Value Approach for Model 5 (Batch 3, 305 and 610 mm).

Predictor Variable	Coefficient	p-Value	Accept Hypothesis if | p | > 0.05
Constant	120.06	0.000	Reject
COVER	-50.056	0.000	Reject
ECR	-1.15	0.794	Fails to Reject
PC_610	20.841	0.000	Reject

Table 7-4f: Statistical p Value Approach for Model 6 (All Specimens).

Predictor Variable	Coefficient	p-Value	Accept Hypothesis if | p | > 0.05
Constant	129.13	0.000	Reject
PC_305	-7.83	0.063	Fails to Reject
PC_610	-29.13	0.000	Reject
PC_813	31.96	0.000	Reject
COVER	-50.95	0.000	Reject
ECR	-0.04	0.989	Fails to Reject

7.2.2 Resultant Models

Model 1 through Model 6 provided a basis to define the significant explanatory variables within each model. T test and p value comparison showed that the reinforcing bar type was insignificant and every other variable appeared to be significant. Thus, the ECR variable was taken out of consideration as a model parameter. In addition, half of the data was randomly selected to define the new corrected models. The other half of the data was then used to check that the defined regression model accurately represented the degree of cracking observed in the specimens. As a result, the following corrected surface cracking models were defined:

Corrected Model 1C: **D = 152 – 63.5(COVER)**

Corrected Model 2C: **D = 56.6 – 25(COVER)**

Corrected Model 3C: **D = 42.9 – 20.9(COVER)**

Corrected Model 4C: **D = 196 – 86.4(COVER)**

Model 1C through 4C are simply the statistically corrected models that provide a measure of the degree of cracking observed in each specimen at 6 months of age. The models presented here support the work done by Dakhil, showing that the concrete clear cover depth largely governs the degree of specimen cracking. However, the models provide no information on the degree of cracking that might be caused by variations in bar spacing, concrete cement type, or boundary conditions present. The influence of bar spacing, concrete cement type and boundary conditions has been presented in the following models.

- Model RM1 was defined to quantify the effect of boundary conditions on surface cracking of concrete bridge decks at 6 months of age. Similar to Model 5, RM1 was defined from the 305 mm (12 in.) and 610 mm (24 in.) size specimens cast from concrete Batch 3 (PC). This model however is quadratic and includes only the variables found to be significant: concrete clear cover and specimen size (boundary conditions). Similar to the previous models, RM1, was defined from a random selection of half of the data. The other half of the data will be used to justify application of the model.

Cracking Model RM1:

$$\mathbf{D = 126 - 23.1(PC\_610) - 53.0(COVER) + 27.9(COVER - 2)^2}$$

- Model RM2 was defined to quantify the effect that reinforcement bar spacing has on the surface cracking of concrete bridge decks at 6 months of age. Resultant model, RM2, was defined to develop a comparison between the 813 mm (32 in.) square specimens cast from Batch 2 (PC), and the 610 mm (24 in.) specimens cast from Batch 3 (PC). It should be noted that this model does not account for batch to batch variations that may have existed. Batch variations (slump and temperature) were however minimal. Similar to the other defined models, RM 3 consists of only the variables found to be significant and was defined using a random selection of half of the data. The variables defined in RM2 are the concrete clear cover and the reinforcing bar spacing. The 813 mm specimens had a 200 mm (8 in.) bar spacing factor and the 610 mm specimens had a 150 mm (6 in.) bar spacing factor.

  Cracking Model RM2:

  $$\mathbf{D = 52.5 - 5.55(PC\_813) - 23.2(COVER)}$$

- Model RM3 was defined to quantify the effect that concrete cement type has on the surface cracking of concrete bridge decks. Resultant model, RM3, was defined to develop a comparison between the 813 mm (32 in.) square specimens cast from Batch 1 (GGBFS), and the 813 mm (32 in.) specimens from Batch 2 (PC). Similar to RM2, it should be noted that this model does not account for batch to batch variations that may have existed. Resultant model RM 3 consists of only the variables found to be significant and was defined using a random selection of half of the data. The variables defined in RM3 are the concrete clear cover and the type of cement binder used in the concrete.

Cracking Model RM3:

D = 102 +35.5(GGBFS_813) – 53.5(COVER)

7.2.2.1 Statistical p Value Comparison

Although the variables used to construct the resultant models were previously found to be significant, out of completeness, a statistical p value comparison was conducted to determine if the model parameters in the multiple linear regression model are significant. The computed p value for each model parameter was less than the level of significance (0.05) for every multiple linear regression model except for resultant model RM1. Thus, each variable in each of the resultant regression models were found to be significant except for the $(COVER – 2)^2$ term in resultant model RM 1. This variable was found to be insignificant, having little effect on the estimation of the degree of cracking. The p-value test results for the resultant models are presented in tables 7-5a through 7-5g.

Table 7-5a: Statistical p Value Approach for CM1 (Batch 3, 305 mm).

Predictor Variable	Coefficient	p-Value	Accept Hypothesis if | p | > 0.05
Constant	151.58	0.000	Reject
COVER	-63.55	0.000	Reject

Table 7-5b: Statistical p Value Approach for CM2 (Batch 3, 610 mm).

Predictor Variable	Coefficient	p-Value	Accept Hypothesis if | p | > 0.05
Constant	54.66	0.000	Reject
COVER	-25.3	0.004	Reject

Table 7-5c: Statistical p Value Approach for CM3 (Batch 2, 813 mm).

Predictor Variable	Coefficient	p-Value	Accept Hypothesis if | p | > 0.05
Constant	42.9	0.001	Reject
COVER	-20.9	0.002	Reject

Table 7-5d: Statistical p Value Approach for CM4 (Batch 1, 813 mm).

Predictor Variable	Coefficient	p-Value	Accept Hypothesis if | p | > 0.05
Constant	196	0.000	Reject
COVER	-86.9	0.000	Reject

Table 7-5e: Statistical p Value Approach for RM1 (Batch 3, 305 and 610 mm).

Predictor Variable	Coefficient	p-Value	Accept Hypothesis if | p | > 0.05
Constant	125.9	0.000	Reject
COVER	-53.0	0.000	Reject
PC_610	-23.1	0.001	Reject
$(COVER-2)^2$	27.88	0.339	Fails to Reject

Table 7-5f: Statistical p Value Approach for RM2 (Batch 3, 610 mm and Batch 2, 813 mm).

Predictor Variable	Coefficient	p-Value	Accept Hypothesis if | p | > 0.05
Constant	52.5	0.000	Reject
COVER	23.2	0.000	Reject
PC_813	-23.2	0.003	Reject

Table 7-5g: Statistical p Value Approach for RM3 (Batch 1, 813 mm and Batch 2, 813 mm).

Predictor Variable	Coefficient	p-Value	Accept Hypothesis if | p | > 0.05
Constant	102	0.000	Reject
COVER	-53.5	0.000	Reject
GGBFS_813	35.5	0.000	Reject

7.2.2.2 Coefficient of Multiple Determination, R^2

Table 7-6 presents the coefficient of multiple determination, R^2, with and without adjustment for the degrees of freedom for the corrected models and the resultant models. The R^2 value simply represents the proportion of variation in specimen cracking that is explained by the predictor variables. The adjusted R^2 reports the R^2 value that would be calculated regardless of the sample size present. As an example, regression model CM1 is able to explain 66.3 percent of the variability observed in the data. Regression Model CM4 (GGBFS, Batch 1) was seen to have the greatest R^2 value being able to explain 85.0 percent of the variation observed in the data.

Table 7-6: Coefficient of Multiple Determination, R^2

Model Number	R^2	R^2 Adjusted
CM1	68.4	66.3
CM2	66.3	62.1
CM3	70.9	67.2
CM4	86.9	85.0
RM1	68.3	64.0
RM2	73.3	70.2
RM3	84.6	82.9

7.2.3 Comparisons between Calculated and Actual Specimen Cracking Values

Regression models CM1 through CM4 were used to compute the modeled degree of cracking for each specimen type. Table 7-7 presents a comparison of the computed values with the observed average degree of cracking values; as expected, the computed values are shown to be comparable. In addition, the modeled degree of cracking determined from RM1 through RM3 have been presented in Table 7-7. The comparison may be seen more clearly in Figures 7-1a through 7-1d. It is clear that the computed regression models are in agreement with the randomly selected cracking values that were not inclusive in the development of the regression model.

Table 7-7: Comparison of Actual and Computed Degree of Cracking Values.

Batch #	Mixture Type	Square Form Size, mm, (in.)	Bar Spacing, mm, (in.)	Concrete Cover, mm, (in.)	Observed Percent Cracked	CM Model	RM Model	Dakhil's Model
						CM4	RM3	
1	PC/GGBS	813 (32)	200 mm (8)	38 (1.5)	62.2	66.4	57.3	N/A
1	PC/GGBS	813 (32)	200 mm (8)	44 (1.75)	35.7	44.8	43.9	N/A
1	PC/GGBS	813 (32)	200 mm (8)	51 (2)	21.9	23.2	30.5	N/A
						CM3	RM2	
2	PC	813 (32)	200 mm (8)	38 (1.5)	13.7	11.6	21.8	N/A
2	PC	813 (32)	200 mm (8)	44 (1.75)	8.4	6.3	8.4	N/A
2	PC	813 (32)	200 mm (8)	51 (2)	1.8	1.1	-5.0	N/A
						CM2	RM2	
3	PC	610 (24)	150 mm (6)	38 (1.5)	16.5	18.4	27.0	N/A
3	PC	610 (24)	150 mm (6)	44 (1.75)	12.8	12.0	13.0	N/A
3	PC	610 (24)	150 mm (6)	51 (2)	3.8	5.6	-1.0	N/A
						CM1	RM1	
3	PC	305 (12)	150 mm (6)	38 (1.5)	58.0	56.8	48.2	81.00
3	PC	305 (12)	150 mm (6)	44 (1.75)	28.5	40.9	33.3	63.52
3	PC	305 (12)	150 mm (6)	51 (2)	3.5	25.0	21.7	45.15
3	PC	305 (12)	150 mm (6)	57 (2.25)	6.9	9.1	13.7	27.11
3	PC	305 (12)	150 mm (6)	64 (2.5)	0.0	-6.8	9.2	10.52

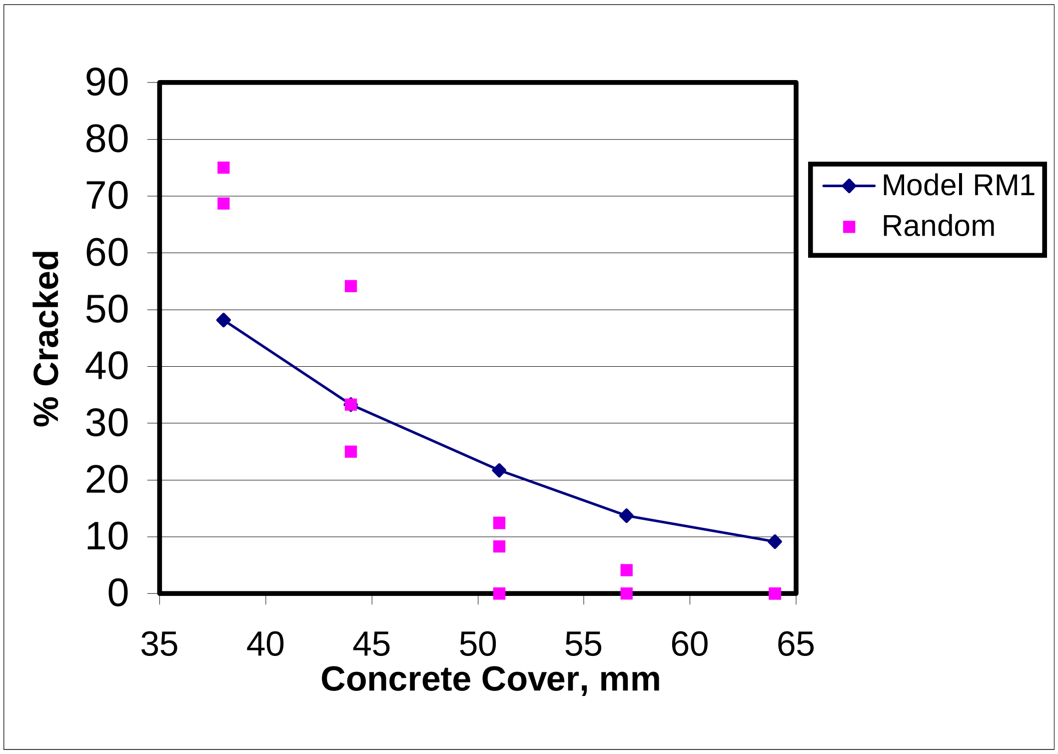


Figure 7-1a: Comparison of Actual Versus Computed Degree of Specimen Cracking (Model RM1). Batch 3 (PC), 305 mm (12 in.) Square Specimen Size

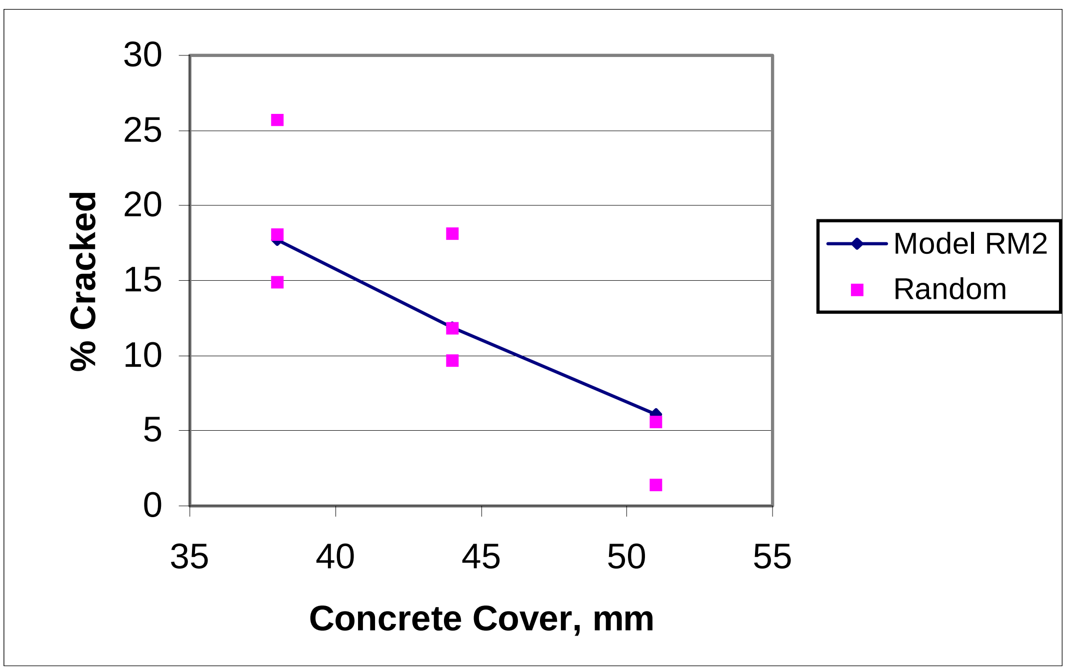


Figure 7-1b: Comparison of Actual Versus Computed Degree of Specimen Cracking (Model RM2). Batch 3 (PC), 610 mm (24 in.) Square Specimen Size, 150 mm (6 in.) Bar Spacing.

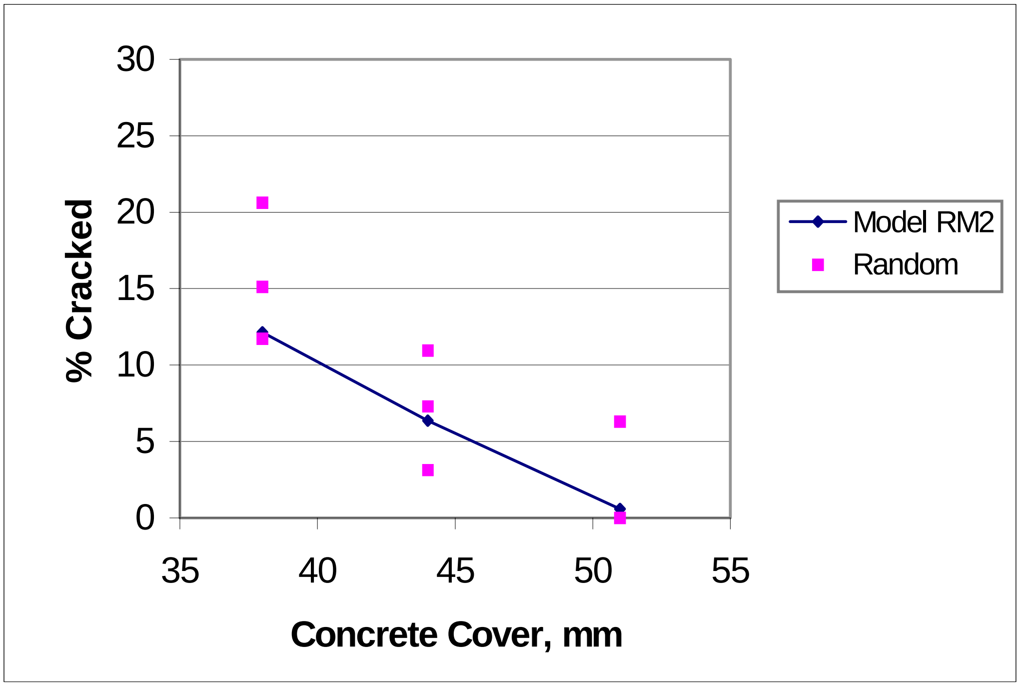


Figure 7-1c: Comparison of Actual Versus Computed Degree of Specimen Cracking (Model RM2). Batch 2 (PC), 813 mm (32 in.) Square Specimen Size, 200 mm (8 in.) Bar Spacing.

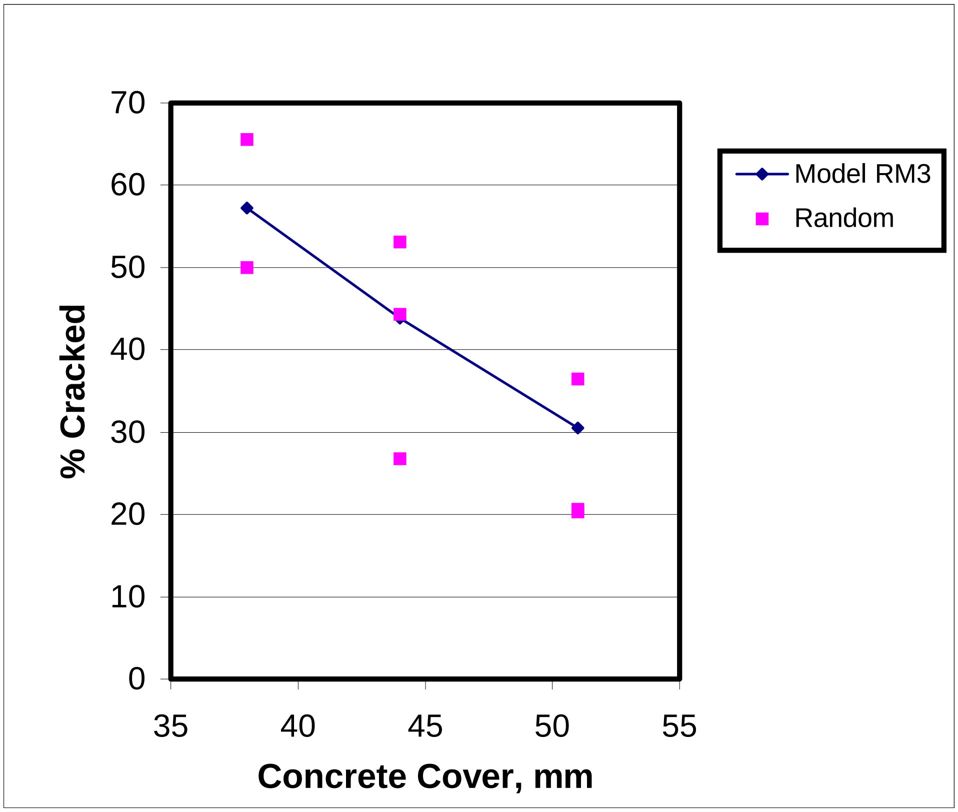


Figure 7-1d: Comparison of Actual Versus Computed Degree of Specimen Cracking (Model RM3). Batch 1 (PC/GGBFS), 813 mm (32 in.) Square Specimen Size, 200 mm (8 in.) Bar Spacing.

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7.2.4 Comparisons of Results to Dakhil's Subsidence Cracking Model

Table 7-7 and Figure 7-2 present the results of actual and computed cracking seen in the 305 mm (12 in.) square specimens over a range of clear cover depths as compared to the model developed by Dakhil (1973).

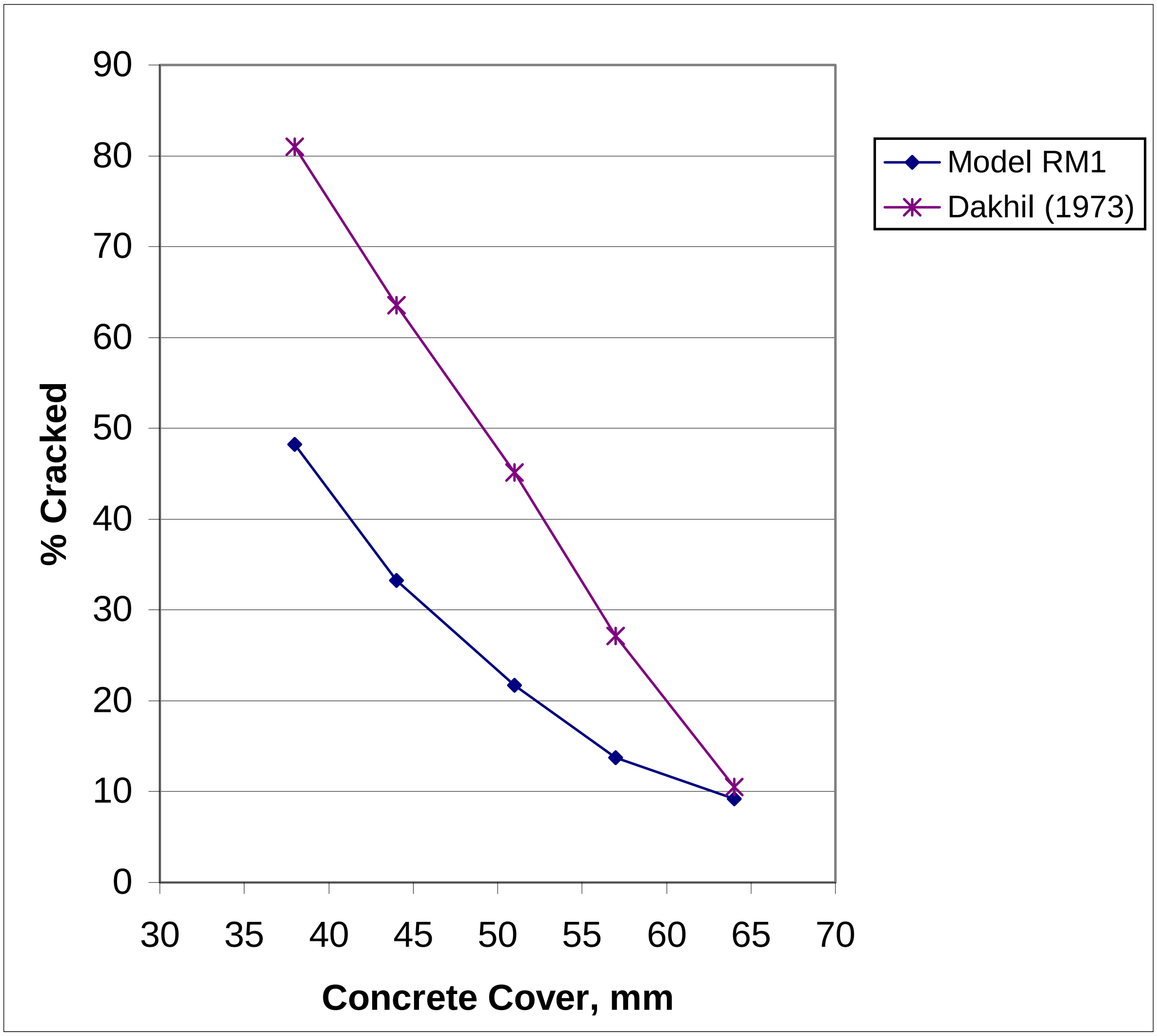

Figure 7-2: Comparison of Actual and Computed Results with the Regression Model Reported by Dakhil. Batch 3 (PC), 305 mm (12 in.) Square Specimen Size.

The probability model established by Dakhil at 6 hours of age exhibit slightly a slightly different linear behavior. Cracking values tend to be 30 percent greater at clear cover depths of 38 mm and only slightly greater at clear cover depths of 64 mm. The variations in the two models may be explained by differences in concrete mixture designs, specimen preparation techniques, and concrete age. Table 7-8 presents notable differences that must be considered before a viable comparison can be made.

Table 7-8: Notable Variations Between the Current Results and the Results from Dakhil.

Notable Variations	Current Study	Dakhil's Study
Concrete Mixture		
W/C Ratio	0.45	0.48
Nominal maximum aggregate size	25 mm (1 in.)	38 mm (1.5 in.)
Concrete Treatment		
Screeding technique	Wooden 2 x 4 fitted with vibrator	Wooden 2 x 4
Finishing technique	Magnesium float	Wet burlap drag
Concrete Specimen Age	6 months	6 hours
Form of Data Utilized	Quantitative	Qualitative

The specimens used in Dakhil's study were cast from concrete that had a w/c ratio of 0.48 and a nominal maximum aggregate size of 38 mm (1.5 in.). The specimens from this study were composed of concrete that had a w/c ratio of 0.45 and a nominal maximum aggregate size of 25 mm (1 in.). Although the concrete was consolidated similarly, the specimens from the two studies were screeded and finished differently.

The concrete surface of the specimens in this study was struck off using a 2 x 4 fitted with a vibrator, and then finished with a magnesium float. Concrete specimens prepared by Dakhil were simply screeded with a 2 x 4 and then finished with a wet burlap drag. These slight differences in the concrete mixture, screeding and finishing technique may influence the degree and rate of subsidence cracking present in the specimens. It is likely that increases in the w/c ratio, and nominal maximum aggregate size would increase the probability for subsidence cracks to develop resulting from possible increase in settlement from additional bleeding.

If further settlement of the plastic concrete resulted from the additional vibration of the concrete surface, the net result would be a densification of the uppermost layer or “skin” of the concrete. This compaction of the surface concrete may release subsidence stresses at the uppermost surface reducing the risk for cracks to appear at an early age. Finishing with a magnesium float may also contribute to a densification of the concrete surface.

In addition to the variations presented here, it is very important to understand that Model RM1 was established from cracking values observed at 6 months of age, whereas, the model established by Dakhil was derived from short term (6 hours) cracking observations. It is highly probable that specimen cracking in this study may have been undetected at early ages due to the narrow crack widths. As specimens underwent drying shrinkage at later ages, the cracks previously formed by subsidence stresses may have opened to widths detectable with the unaided eye. Specimen cracking was found to be uniform over the reinforcing bar, consistent with subsidence observations made by Dakhil.

It should also be made clear that the cracking models in this study were defined by quantitative data, which consisted of the percent of cracking observed in each specimen. Whereas, the probability model established by Dakhil was defined from qualitative data in which no crack lengths were measured.

7.3 Effect of Epoxy Coated Reinforcement on Subsidence Cracking

Although some differences in the degree of cracking became visible at various times throughout the study, specimens cast with ECR exhibited similar cracking to the BRS specimens. Statistical t test and p-value analyses confirm that there is no significant difference between specimens cast with ECR and BRS.

7.4 Effect of Cement Type on Subsidence Cracking

Specimens cast with the PC/GGBFS blended mixture of Batch 1 were compared to the specimens cast with PC from Batch 2. The cement mixture type is the only significant difference between the test specimens. Although possible batch to batch variations may have existed between the two mixes, all other experimental variables were held constant (bar spacing, form size, initial slump).

Specimens cast with the PC/GGBFS blended cement mixture exhibited the greatest degree of cracking for the initial and extended study. At 6 hours of age, specimens cast with the PC/GGBFS mixture, having a clear cover depth of 38 mm (1.5 in.), exhibited average cracking values of 4.3 percent, whereas similar specimens cast with PC experienced no observable cracking. This cracking relationship is more apparent at 180 days of age; specimens cast with the same PC/GGBFS mixture exhibited cracking values as high as 62.2 percent, whereas similar specimens cast with PC (Batch 2) experienced only 13.7 percent. The combined regression model RM3 accommodates for this relationship by adding 35.5 percent to βo. In this model, βo represents the degree of cracking value for cracking observed in the PC, 813 mm square specimens from Batch 2. Thus, the model shows that the specimens cast with the PC/GGBFS blended cement mixture had a 35 percent higher chance for surface cracking. This difference in the degree of cracking may be attributable to the differing curing demands required for the two cement mixture types. In general, slag cements gain strength at later ages and are more sensitive to reduced curing regimens.

7.5 Effect of Boundary Conditions on Subsidence Cracking

The 305 and 610 mm (12 and 24 in.) square PC specimens cast from Batch 3 are directly comparable with respect to their boundary conditions. The 610 mm specimens contained a total of 5 bars, 3 top reinforcing bars, and 2 bottom temperature bars, tied at each intersection. The 305 mm specimens had only one steel reinforcing bar. For each clear cover, the 305 mm square specimens exhibited more cracking than the 610 mm specimens. At 30 days of age, the average degree of cracking for the 38 mm (1.5 in.) clear cover was 23.7 percent for the 305 mm square specimens, whereas there was no observable cracking in the 610 mm specimens. At 180 days of age, the average degree of cracking was 58 percent for the 305 mm square specimens, and only 16.5 percent for the 610 mm specimens. This relationship is visible in the 180-day linear regression model RM1. The coefficient for the 610 mm specimen size has a value of – 23.1 percent. This means that the 610 mm square specimens experienced 23.1 percent less cracking on average than the 305 mm specimens. Thus, it is clear that specimens cast with a single bar have a greater tendency for cracking. The cracking relationship may be best explained by the temperature and shrinkage bottom reinforcing bars. Lower reinforcing bars may provide a dispersion of subsidence stress during placement and then provide physical restraint capable of limiting cracking at later ages.

Although possible batch to batch variations existed between the 813 and 610 mm (32 and 24 in.) specimens cast with the PC concrete mixtures, Batch 2 and Batch 3 are comparable with respect to their boundary conditions. The 813 mm square specimens from Batch 2 had a bar spacing of 200 mm (8 in.), and the 610 mm square specimens had a bar spacing of 150 mm (6 in.). Figures 6-5 through 6-7 show that the degree of cracking was only slightly different between the specimens cast from Batch 2 and Batch 3. The average (ECR and BRS) degree of cracking at 30 days were low; specimens cast with Batch 2 with a concrete clear cover of 38 mm (1.5 in.) were 4.1 percent cracked. Whereas, the 610 mm specimens from Batch 3 exhibited no cracking. At 180 days, cracking values were 13.7 percent from Batch 2 and 16.5 percent from Batch 3. These small differences in cracking percentages between the 813 and 610 mm (36 and 24 in.) square specimens were found to be statistically significant, however, the influence of reinforcing bar spacing on regression model RM2 was found to be minimal. The defined regression model RM2 accounts for this variation by subtracting 5.55 percent from β_o, in which β_o is the degree of cracking present in the 610 mm specimens. Thus, specimen cracking appears to reduce when reinforcing bar spacing is increased. This finding however conflicts with previous relationships observed by Weyers et al. (1982). Weyers et al., using a photoelastic medium, demonstrated that tensile stresses above rigid inclusions increase as a function of increases in the reinforcing bar spacing factor [Weyers et al, 1982]. It is possible that the small reduction in cracking observed in the PC 813 mm specimens is due to batch to batch variations and or differences in form size.

7.6 Effect of Variations in Specimen Preparation Conditions

7.6.1 Concrete Slump:

Specimens cast with the PC/GGBFS blended cement mixture (Batch 1) exhibited the highest degree of cracking and the smallest loss of slump during the concrete placement period. Concrete from Batch 1 began with a slump of 89 mm (3.5 in.) and finished with a slump of 76 mm (3.0 in.), a loss of 13 mm (0.5 in.). The PC concrete mixture (Batch 2) began with a slump of 83 mm (3.5 in.) and finished at a slump of 57 mm (2.5 in.), a loss of about 25 mm (1 in.). This experimental difference in slump values between Batch 1 and 2 is small. Increases in subsidence cracking observed in the PC/GGBFS blended cement mixture may have resulted from its longer retention of slump and slower hydration rate. Batch 3 began with a slump of 152 mm (6 in.), and ended with a slump of 127 mm (5 in.). According to Dakhil et al. (1975), Batch 3's significantly higher slump should have exhibited subsidence cracking slightly higher than similar mixes with lower slumps. However, this relationship was not clearly seen from the results; the 610 mm (24 in.) PC specimens and the 810 mm (32 in.) PC specimens had a similar degree of cracking. This may be seen in Figures 6-5 through 6-7.

7.6.2 Concrete Curing Conditions:

Compared to other specimens, those cast with the PC/GGBFS blended cement mixture (Batch 1) experienced the most severe exposure conditions. As Table 6-2 shows, the ambient temperature was 30.8 °C (87.4°F) for specimens cast with Batch 1, 25.0 °C (77.0°F) for specimens cast with Batch 2, and 21.7 °C (71.1°F) for specimens cast with Batch 3. Also, the relative humidity was lower for specimens cast with Batch 1, with little wind present. The elevated temperatures, lower relative humidity, and little wind may have increased the likelihood of early stage cracking on the surface of the PC/GGBFS specimens.

7.7 Discussion of Experimental Error Possible

The following discussion outlines the types of error that might have occurred during the study period. Possible error may have resulted from errors in construction, concrete treatment, measurement, and/or instrumentation.

7.7.1 Construction Errors

During the construction of the specimens, slight errors may have occurred in the placement of the steel bars at the proper clear cover and or bar spacing. While errors in the bar spacing would be fairly insignificant, errors in the reinforcing bar clear cover would greatly impact the results of this study. Likewise, any small variation in the screeding level may have created undesirable clear cover depths, significantly affecting the results of this study.

7.7.2 Concrete Treatment Error

During the initial study period, the specimens were wet cured continuously with water and left uncovered for 6 hours. During this time, localized drying may have taken place on the specimen top surface, increasing the likelihood for plastic and drying shrinkage. Errors from concrete treatment would have been small, since each specimen was constantly saturated with water.

7.7.3 Measurement Error

Numerous crack observations were made during the initial and extended duration of this study. The difficulty in observation and measurement of these cracks may have increased the margin for error. Crack widths measured in this study were very small; many were so narrow that they almost escaped detection by the human eye. Spraying the specimens with water prior to observation made these cracks more visible.

7.7.4 Instrumentation Error

Improper measurement or calibration of the entrained air content meter and/or compressive test load cell used in this study may have led to instrumentation error. Errors in instrumentation are however unlikely since laboratory equipment were recently calibrated.

8. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions have been made from this study:

- There is not a significant difference in the probability of occurrence of subsidence cracking of concrete between concrete cast with 16 mm diameter (#5) epoxy coated reinforcing steel and bare reinforcing steel bars.
- Concrete clear cover depth has the greatest impact on the probability of occurrence of subsidence cracking.
- Concrete cement type has a significant impact on the probability of occurrence of subsidence cracking.
- Cracks due to concrete subsidence may not become apparent in concrete specimens until later concrete ages.

From the results of this study, the following recommendations have been made:

- Further studies should be conducted to determine the influence of various finishing techniques on the probability of occurrence of subsidence cracking.
- Further research should be conducted on the influence of various cement types on the probability of occurrence of subsidence cracking.

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APPENDIX I

(180 days cracking results)

Batch # 1, (GGBFS/PC), 813 mm Square Specimen Size

Treatment	Measurements			% Cracked	S. DEV.,s^2	Crack Width, mm
Bare, 1.5 in.	60.9	69.3	50.0	60.1	9.665	0.07
Bare, 1.75 in.	48.4	22.4	19.3	30.0	16.014	0.05
Bare, 2.0 in.	20.6	15.6	24.2	20.1	4.313	0.05
ECR, 1.5 in.	71.9	65.6	55.7	64.4	8.141	0.06
ECR, 1.75 in.	44.3	26.8	53.1	41.4	13.383	0.05
ECR, 2.0 in.	20.3	14.1	36.5	23.6	11.557	0.05

Batch # 2, (PC), 813 mm Square Specimen Size

Treatment	Measurements			% Cracked	S. DEV.,s^2	Crack Width, mm
Bare, 1.5 in.	8.9	15.1	15.6	13.2	3.768	0.05
Bare, 1.75 in.	2.6	10.9	21.4	11.6	9.394	0.05
Bare, 2.0 in.	0.0	0.0	6.3	2.1	3.608	0.05
ECR, 1.5 in.	10.4	11.7	20.6	14.2	5.526	0.05
ECR, 1.75 in.	3.1	5.2	7.3	5.2	2.083	0.05
ECR, 2.0 in.	0.0	0.0	4.7	1.6	2.706	0.05

Batch # 3, (PC), 610 mm Square Specimen Size

Treatment	Measurements			% Cracked	S. DEV.,s^2	Crack Width, mm
Bare, 1.5 in.	25.7	9.7	18.1	17.8	7.989	0.05
Bare, 1.75 in.	0.0	11.8	20.5	12.2	10.247	0.05
Bare, 2.0 in.	5.6	0.7	1.4	2.8	4.811	0.05
ECR, 1.5 in.	14.9	15.3	15.3	15.2	0.200	0.05
ECR, 1.75 in.	9.7	18.1	16.7	14.8	4.465	0.05
ECR, 2.0 in.	5.6	6.3	3.5	2.1	3.608	0.05

Batch # 3, (PC), 305 mm Square Specimen Size

Treatment	Measurements			% Cracked	S. DEV.,s^2	Crack Width, mm
Bare, 1.5 in.	75.0	33.3	68.8	59.0	22.470	0.05
Bare, 1.75 in.	33.3	25.0	25.0	27.8	4.811	0.05
Bare, 2.0 in.	0.0	0.0	8.3	2.8	4.811	0.05
Bare, 2.25 in.	0.0	4.2	37.5	13.9	20.554	0.05
Bare, 2.5 in.	0.0	0.0	0.0	0.0	0.000	0.05
ECR, 1.5 in.	75.0	16.7	79.2	56.9	34.944	0.05
ECR, 1.75 in.	33.3	0.0	54.2	29.2	27.323	0.05
ECR, 2.0 in.	0.0	0.0	12.5	4.2	7.217	0.05
ECR, 2.25 in.	0.0	0.0	0.0	0.0	0.000	0.05
ECR, 2.5 in.	0.0	0.0	0.0	0.0	0.000	0.05

Batch # 1, (GGBFS/PC), 813 mm Square Specimen Size						
Clear Cover, in.	1.5	1.75	2		Random Data	
Clear Cover, mm	38	44	51		x	y
	60.9	48.4	20.6		38	50
	69.3	22.4	15.6		38	65.6
	50.0	19.3	24.2		44	44.3
	71.9	44.3	20.3		51	36.5
	65.6	26.8	14.1		44	26.8
	55.7	53.1	36.5		44	53.1
Cracking Mean,%	62.24	35.72	21.88		51	20.6
Std. Dev.	8.34	14.59	8.03		51	20.3
COV,%	0.13	0.41	0.37			
Confid.	6.67	11.68	6.43			
RM3 Half Model, %	57.25	43.88	30.50			
Full Model, %	53.10	40.05	27.00			
Specific Model,%	59.95	39.78	19.60			
CM4 Half Specific, %	66.40	44.80	23.20			

Batch # 2, (PC), 813 mm Square Specimen Size						
Clear Cover, in.	1.5	1.75	2		Random Data	
Clear Cover, mm	38	44	51		x	y
	8.9	2.6	0.0		38	15.1
	15.1	10.9	0.0		38	11.7
	15.6	21.4	6.3		38	20.6
	10.4	3.1	0.0		44	10.9
	11.7	5.2	0.0		44	3.1
	20.6	7.3	4.7		44	7.3
Mean	13.7	8.4	1.8		51	0
Std. Dev.	4.27	7.03	2.87		51	6.3
COV,%	0.31	0.83	1.57			
Confid.	3.42	5.62	2.29			
RM2 Half Model, %	12.15	6.35	0.55			
Full Model, %	21.1	8.1	-5.0			
Specific Model,%	13.9	7.95	2			
CM3 Half Specific, %	11.55	6.325	1.1			

APPENDIX I (Cont.)

Batch # 3, (PC), 610 mm Square Specimen Size						
Clear Cover, in.	1.5	1.75	2		Random Data	
Clear Cover, mm	38	44	51		x	y
	9.7	0.0	5.6		38	18.1
	25.7	11.8	0.7		38	25.7
	18.1	20.5	1.4		38	14.9
	14.9	9.7	5.6		44	11.8
	15.3	18.1	6.3		44	9.7
	15.3	16.7	3.5		44	18.1
Mean	16.5	12.8	3.8		51	5.6
Std. Dev.	5.26	7.43	2.35		51	1.4
COV,%	0.32	0.58	0.62			
Confid.	4.21	5.94	1.88			
RM2 Half Model, %	17.7	11.9	6.1			
Full Model, %	23.05	10.525	-2			
Specific Model,%	17.35	11.025	4.7			
CM2 Half Specific, %	18.35	11.975	5.6			

Batch # 3, (PC), 305 mm Square Specimen Size							
Clear Cover, in.	1.5	1.75	2	2.25	2.5	Random Data	
Clear Cover, mm	38	44	51	57	64	x	y
	75.0	33.3	0.0	0.0	0.0	38	75
	33.3	25.0	0.0	4.2	0.0	38	68.8
	68.8	25.0	8.3	37.5	0.0	44	25.0
	75.0	33.3	0.0	0.0	0.0	44	33.3
	16.7	0.0	0.0	0.0	0.0	44	54.2
	79.2	54.2	12.5	0.0	0.0	51	8.3
Mean	58.0	28.5	3.5	6.9	0.0	51	0.0
Std. Dev.	26.30	17.56	5.54	15.06	0.00	51	0
COV,%	0.45	0.62	1.59	2.17	0.00	51	12.5
Confid.	21.04	14.05	4.43	12.05	0.00	57	4.2
RM1 Half Model, %	52	38	24	10	-4	57	0
Full Model, %	43.85	31.325	18.8	6.275	-6.25	64	0
Specific Model,%	46.5	32.75	19	5.25	-8.5	64	0
CM1 Half Specific, %	56.75	40.875	25	9.125	-6.75	64	0
Quadratic	48.2	33.3	21.7	13.7	9.2		
	Y = 126 - 23.1 PC_610 - 53.0 COVER + 27.9 Quadratic						

APPENDIX II

VDOT Class 30 (A4) Concrete Mixture Designs

Mixture Design and Handling	Mixture 1	Mixture 2	Mixture 3	Details/Source
Mix Design Number	210-8-00	210-8-00	210-8-00	
Date Delivered	7/10/00	7/11/00	7/13/00	
Amount Ordered, cuy.	4	4	3.5	
Mix Time, AM	9:26	10:17	10:07	
Delivery Time, AM	10:10	10:35	10:40	
Designed Air Content, %	6.5+1-1.5	6.5+1-1.5	6.5+1-1.5	
Designed Slump, in.	2-4	2-4	2-4	
Mixture Proportions				
Natural Sand, lb.	1221	1221	1221	Wythe Stone
No. 57 N.P. Coarse Aggregate, lb.	1635	1635	1635	Salem Stone
Type I/Type II Portland Cement, lb.	318	635	635	Capitol Cement
Pozzolans, lb.	317	0	0	Blue Circle Cement
Water, lb.	286	286	286	Municipal
Air Entrainment Admixture, oz.	2 to 5	2 to 5	2 to 5	Sika Corp.
Retarder Admixture, oz.	18 to 22	18 to 22	18 to 22	Sika Corp.

APPENDIX III

1997 Virginia Department of Transportation (VDOT) Road and Bridge Specifications

Various sections of the VDOT Metric Road and Bridge Specifications were written to minimize bridge deck damage due to construction practices. In the following section, applicable excerpts from the specification have been included. Requirements on concrete design characteristics, control techniques, clear cover, reinforcement, forms, consolidation, finishing and curing were considered during the formulation of this research plan.

Concrete characteristics: *(Sect. 217.07, pp. 195-196)*

- Prior to mixing concrete, the Contractor shall submit, or shall have his supplier submit, for approval concrete mixture design(s) conforming to the specifications for the class of concrete specified.
- The Contractor shall furnish and incorporate an approved water-reducing and retarding admixture in bridge deck concrete and in other concrete when conditions are such that the initial set may occur prior to completion of approved finishing operations. An approved water-reducing admixture shall be furnished and incorporated in concrete when necessary to provide the required slump without exceeding the maximum water/cement. (VDOT Table II-17 for Class 30 concrete requires a concrete slump of 50-100 and 50-175 for concrete containing water reducers.)
- Concrete shall be air entrained. The air content shall conform to the requirements of VDOT Table II-17. (Class 30 concrete pavements require an air content of 6.5 ± 1.5 % or 7.5 ± 1.5 % for concrete containing water reducers.)

Concrete control tests: (Sect. 217.08, pp. 189)

- Air and consistency tests will be performed by the department prior to discharge into forms to ensure that specification requirements are consistently being complied with for each class of concrete.
- Entrained air content will be determined in accordance with the requirements of AASHTO T152 or T196.

Concrete clear cover: *(Sect.406.03 pp. 481-482)*

- In substructures, cover over the top mat of reinforcement shall be at least 65 mm.
- In corrosive or marine environments or under other severe exposure conditions, the minimum cover shall be increased 25 mm except where epoxy-coated reinforcement is used.
- Bars shall be placed so that the concrete cover as indicated on the plans will be maintained within a tolerance of 0 to +13 mm in the finally cast concrete.

Steel reinforcement: *(Sect. 223.02 pp. 209-210 and 406.03 pp. 481)*

- Longitudinal bars for continuous reinforced concrete pavement shall conform to the requirements of ASTM A615M, Grade 400.
- Handling and storage of the coated bars shall conform to the requirements of AASHTO M284.
- Visible damage to the epoxy coating shall be patched or repaired with materials compatible to the existing coating in accordance with AASHTO M284.
- Tie Wires used with epoxy-coated steel shall be plastic coated or epoxy coated.

Concrete forms: *(Sect. 404.03 pp. 439-440)*
- On concrete beam bridges, the Contractor shall have the option of using corrugated metal bridge deck forms, prestressed deck panels, or wood forms to form that portion of bridge decks between beams unless otherwise specified on the plans.
- Forms shall be treated with an approved oil or form-coated material or thoroughly wetted with water immediately before concrete placement.

Consolidation: *(Sect. 404.03, pp. 449 and 316.04 pp. 395)*
- Vibration shall be internal to the concrete but not applied directly to reinforcement or formwork.
- The intensity of vibration shall visibly affect a mass of concrete over a radius of at least 450 mm.
- Vibration shall be of sufficient duration and intensity to consolidate concrete thoroughly but shall not be continued so as to cause segregation.
- Concrete shall be thoroughly consolidated against forms and joint assemblies by means of full-width vibration. Vibrators will not be permitted to come in contact with a joint assembly, reinforcement or side of forms. The vibrator shall not be operated for more than 15 seconds in any one location.

Concrete finish (floating): *(Sect. 316.04, pp. 399-400)*
- Hand finishing will be permitted only under the following conditions: (1) to finish concrete already deposited on the grade in the event of a breakdown of mechanical equipment; and (2) to finish narrow widths, approach slabs of irregular dimensions where the operation of mechanical equipment is impractical.
- Hand method: Following strike off by an approved screed, concrete shall be smoothed with a darby to level raised spots or fill depressions. Long-handled floats or hand floats of wood or metal, as the area dictates, may be used in lieu of darbies to smooth and level concrete surface. Excessive bleed water shall be wasted over the side forms after each pass of the float.

Concrete finish (texture): *(Sect. 316.04, pp. 401and 404.03, pp. 451)*
- Prior to grooving, multi-ply damp fabric shall be dragged over the pavement surface to provide a gritty texture on ridges between grooves.
- Placing and consolidating concrete shall be conducted to form a compact, dense, impervious mass of uniform texture that will show smooth faces on exposed surfaces.

Concrete curing: *(Sect. 316.04, pp. 401 and 404.03, pp.457)*
- Care shall be taken in hot, dry, or windy weather to protect the concrete from shrinkage cracking by applying curing medium at the earliest possible time after

finishing operations and after the sheen has disappeared from the surface of the pavement.

- Water used for curing concrete shall be clean, clear, and free from oil and other deleterious substances and shall have a pH of at least 4.5.
- Concrete shall be cured for 7-days, regardless of the strength obtained with control cylinders. During this 7-day curing period, a curing agent or medium shall be used.

Concrete protection: *(Sect. 404.03, pp. 457-458)*

- Concrete shall be protected from rain.
- Concrete shall be protected from freezing by approved coverings.
- Protection shall be provided to prevent rapid drying of concrete as a result of low humidity, high wind, high atmospheric temperatures or combinations thereof. If the maximum evaporation rate, as determined from Figure 1 of ACI 308-2 exceeds 0.5 kg/m2/hr for Class 30 concrete bridge deck placement or 0.3 kg/m2/hr for latex modified concrete overlays and other hydraulic cement overlays with a water/cement ratio of 0.40 or less, the Contractor shall use protective measures to prevent shrinkage cracking.
- After concrete in finished surfaces has begun to set, it shall not be walked on or otherwise disturbed for at least 24 hours.

Nathan L. Kyle

Vita Highlights

For the past six years, I have been studying Civil Engineering at Virginia Tech. I worked three semesters for Norfolk Southern as a co-op student in the Facilities and Equipment Department. As an undergraduate, I was a member of the Virginia Tech chapter ASCE and Chi Epsilon honor fraternity. I was involved with the ASCE steel bridge and concrete canoe design competitions. Serving two years as co-chair for the form construction committee for the ASCE concrete canoe team, I had the opportunity to travel to Florida and Colorado for national competitions. As a senior, I performed an independent study on "The corrosion inhibition of a migrating corrosion inhibitor in salt laden environments." I began my graduate degree in January of 2000 while dual enrolled as an undergraduate. My graduate research included a study of "Subsidence cracking of concrete over steel reinforcement bar in bridge decks." I graduated in May of 2001 with a Masters of Science in Civil Engineering and then married in late May to a lovely young lady, Miss Emily Beth Hutchings.