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Curing Practices to Reduce Plastic Shrinkage in Concrete Bridge Decks

Steven W. Peyton, Chris L. Sanders,
Sergio G. Arratia, W. Micah Hale

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7. Authors Steven W. Peyton, Chris L. Sanders, Sergio G. Arratia, and W. Micah Hale			8. Performing Organization Report No. AHTD TRC 0603	
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16. Abstract The large, exposed area of concrete bridge decks makes proper curing critical and difficult. Plastic shrinkage cracks are common in improperly cured bridge decks. The objective of this research was to identify curing regimens that successfully reduce plastic shrinkage cracking. This was accomplished by surveying Arkansas Highway and Transportation Department engineers to determine the current construction practices on Arkansas bridge decks, documenting the curing of five bridge decks under construction, and studying the effectiveness of curing regimens in the laboratory. The survey responses show inconsistencies in the interpretations of the construction specifications that were also evident in the field study. Delayed curing, high evaporation rate, and increased girder deflection increased the likelihood of cracking in the decks. Inconsistent application of curing materials caused increased cracking in the laboratory study. Removing impediments, such as tined finishes, to timely curing and clarifying curing specifications would reduce cracking in bridge decks				
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CHAPTER 1

INTRODUCTION

1.1 The Importance of Reducing Shrinkage Cracking in Bridge Decks

Plastic shrinkage occurs when water evaporates from the surface of freshly placed concrete faster than it is replaced by bleed water from below, producing shrinkage in the top surface of the concrete. Cracking results when the tensile stresses caused by plastic shrinkage overcome the capacity of the weak concrete. Plastic shrinkage cracks are common in improperly cured bridge decks. Drying shrinkage, also one of the main causes of deck cracking (Xi et al, 2000), is caused by the loss of water from hardened concrete exposed to air at a relative humidity less than 100 percent. Shrinkage cracking compromises the long-term durability by allowing water and damaging chemicals into the interior region where they can attack the concrete and reinforcement.

1.2 The Importance of Curing for Concrete Bridge Decks

Ineffective curing is reported as the most common cause for plastic shrinkage cracking (Russell, 2004). ACI Committee 116 (1967) defines curing as the "maintenance of humidity and temperature of freshly placed concrete during some definite period following placing, casting or finishing to assure satisfactory hydration of the cementitious materials and proper hardening of the concrete". The large exposed area of bridge decks makes proper curing more critical and more difficult. Although the current Arkansas Highway and Transportation Department (AHTD) specifications comply with American Association of State Highway and Transportation Officials (AASHTO) guidelines, bridge deck cracking still occurs. Information regarding the best curing methods to minimize shrinkage cracking in reinforced bridge decks needs to be developed.

1.3 The Importance of Concrete Mixture Proportioning

Changes to mixture proportions can improve both the fresh and hardened properties of concrete. Improvements to fresh concrete properties include reduced plastic shrinkage, autogenous shrinkage, and subsidence of plastic concrete, and increased ease of placement. Improvements to the hardened properties of concrete include decreased drying shrinkage and permeability, and increased durability and strength. These improvements can be achieved by changing the amount of cement, supplementary cementing materials (SCMs), water content, water-to-cementitious materials ratio (w/cm), aggregate type and gradation, and chemical admixtures.

1.4 Background

1.4.1 Cracking Definitions

While not typically large enough to compromise the structural integrity of the bridge deck, shrinkage cracking compromises the long-term durability by allowing water and damaging chemicals into the interior region where they can attack the concrete and reinforcement. ACI committee 224 recommends that cracks in concrete exposed to deicing salts be limited to 0.008 in. in width (2005). Krauss and Rogalla (1996) report differences in research into crack width's effect on the extents of corrosion in reinforcing steels. However, it is widely held that cracking in bridge decks increases the rate at which moisture and deicing salts reach the reinforcing steel speeding the onset of the corrosion process (Russell, 2004). University of Kansas researchers found that while high quality concrete slows the penetration of chlorides to the reinforcing steel, cracks allow the chloride levels near the steel to reach corrosive levels within the first year (Darwin et al., 2006).

Thus, the study of concrete shrinkage cracking is a common focus in bridge deck research. There are four main types of shrinkage that occur in concrete. Autogenous and plastic shrinkage occur at an early concrete age. Drying shrinkage and carbonation shrinkage occur throughout the entire life of hardened concrete.

Autogenous shrinkage is caused by the consumption of water during the hydration process. The products of the hydration process take up less volume than the water and cement molecules (ACI 224, 2005). Autogenous shrinkage is accompanied by a reduction of the relative humidity in concrete and an increase in the surface tension in capillary water. Autogenous shrinkage occurs without moisture exchange between concrete and the surrounding environment. It is suggested to design concrete mixtures with a water-to-cementitious materials ratio (w/cm) greater than 0.40 to reduce the risk of early cracking due to autogenous shrinkage (Xi et al., 2000).

Drying shrinkage is caused by the loss of water from hardened concrete exposed to air with less than 100% relative humidity. Concrete paste swells in the presence of water. As the hardening concrete dries, either due to evaporation or hydration, the paste shrinks. If the shrinkage strains are greater than the local tensile strength of the concrete, cracks form (ACI 224, 2005). Drying shrinkage is one of the main causes of deck cracking (Xi et al, 2000). The amount of evaporated water is the most important indicator of the final amount of drying shrinkage. Curing, relative humidity, and concrete temperature control the rate of drying shrinkage (Krauss and Rogalla, 1996).

Carbonation shrinkage results from a reaction of carbon dioxide and a hydroxide or oxide within the concrete paste to form a carbonate. The carbonate compounds react with calcium to form calcium carbonate (ACI 116, 1967). Preventing combustion exhaust gases from being directed at the fresh concrete will limit carbonation shrinkage, as atmospheric carbon dioxide does not penetrate beyond ½ in. beyond the surface (ACI 224, 2005).

Plastic shrinkage cracking occurs when water evaporates from the surface of freshly placed concrete faster than it is replaced by bleed water from below. Water bleeds to the surface of the concrete as the solids subside, or settle to the bottom. As the flow of bleed water to the surface slows, evaporation takes over, and drying of the surface can occur. This produces shrinkage in the top surface of the concrete. Restraint forces are created within the wet concrete that is not experiencing the same shrinkage. This generates tensile stresses, which cannot be resisted by the weak, plastic concrete. These stresses produce cracks in the surface (ACI 224, 2005). The cracks can be random (Babaei and Fouladgar, 1997) or parallel and regularly spaced (ACI 224, 2005). Crack depths can be 2 to 3 in. up to full depth, while lengths can range from 2 to 3 ft., and widths from 0.002 to 0.25 in. (Krauss and Rogalla, 1996).

1.4.2 Construction Practices to Reduce Plastic Shrinkage Cracking

Bridge deck cracking is a major concern for transportation agencies across the country (Russell, 2004). Over thirty state transportation agencies have researched the scope, causes, or possible solutions to the problem, with varying success. Shrinkage cracking is a common focus of this research. *NCHRP Synthesis 333* (Russell, 2004) recommends the following construction practices to “enhance the performance of concrete bridge decks”:

- Moderate concrete temperatures at the time of placement;
- Minimize surface evaporation with windbreaks and fogging equipment;
- Minimize required finishing operations;
- Begin wet curing as soon as possible after finishing any portion of the concrete surface;
- Wet cure for 7 days minimum;
- Use curing compound applied post-wet cure to reduce shrinkage.

A review of bridge deck construction and causes of cracking on Colorado Department of Transportation bridges by the University of Colorado at Boulder identified the following recommended changes to their specifications to decrease the incidence of cracking (Xi et al, 2000):

- Reduce the maximum concrete temperature from 90°F to 80°F;
- Measure evaporation rate or estimate using a chart, and avoid placement when rate is greater than 0.20 lb./ft.^{2}/hr.;
- Apply fogging immediately and until final cure is placed;
- Finish and texture the surface as soon as possible, minimizing hand finishing to avoid delay of final cure;
- Map and seal all cracks occurring within the first year after placement.

Missouri Department of Transportation (MoDOT) conducted a Joint Process Review of its policies and procedures for bridge deck construction with the Federal Highway Administration. Their findings recommended several changes to the mixtures and procedures used and an emphasis on pre-placement planning with the contractors. Some of the suggested construction and curing practices were:

- Emphasizing proper consolidation technique before and during the placement;
- Providing training to contractors and MoDOT staff to avoid the over-finishing of deck concrete;
- Implementing post-cure mechanical grooving in lieu of the tined finish;
- Improving the consistency of curing compound and wet-cure applications.

The results of which, based on a limited study of decks using the new policies, was a 75% reduction in cracking. (MoDOT, 2005).

1.4.3 Concrete Properties and Proportioning Guidelines to Reduce Cracking

In a questionnaire for the NCHRP Synthesis 333 (TRB, 2004), 45 agencies that responded reported strategies which they are currently using to minimize cracking in bridge decks. The following list is the strategies (as relating to concrete properties) along with the number of responses out of 45:

- Specify maximum slump, 40;
- Specify maximum concrete temperature, 36;
- Specify maximum cementitious materials content, 15;
- Specify maximum concrete compressive strength, 2.

Research performed by the researchers at the University of Colorado at Boulder and the Colorado Department of Transportation examined the causes of cracking in newly constructed bridge decks in Colorado and identified necessary changes in the material properties, construction processes and design specifications to decrease bridge deck cracking. Recommendations from the research relating to material factors include:

- Use of fly ash at a replacement rate of 20% to 25%;
- Maximum cement content of 470 lb/yd^3;
- w/cm of approximately 0.40;
- Use of silica fume (5% replacement rate) and slag cement;
- Specified strengths at early ages instead of just 28 days;
- Permeability, drying shrinkage and crack resistance tests should be considered as acceptance tests;
- Use of large, well-graded aggregates.

Current research, headed by researchers at the University of Kansas along with 15 state Departments of Transportation and the Federal Highway Administration, is determining best practices to minimize bridge deck cracking. The techniques to reduce cracking include reducing the volume of cement and water, maintaining an adequate air content, improving aggregate gradations, decreasing the importance of high compressive strength, and controlling the fresh concrete temperature during placement. Materials specifications developed from this research include:

- Maximum cement content of 563 lb/yd^3;
- Requiring an “optimized” aggregate gradation (based on the Shilstone method);
- Maximum paste (water and cement) content of 27%;

- An air content of 6.5% to 9.5 %;
- Fresh concrete temperature range of 50°F to 70°F.

1.4.4 AHTD Specifications for Concrete Bridge Decks

Concrete used in bridge decks in Arkansas is governed by Section 802 of the *Standard Specification for Highway Construction* (2003) and is classified as Class S(AE) concrete (AE for air entrained). For Class S(AE) concrete, AHTD requires a minimum 28-day compressive strength of 4000 psi, a slump of 1 to 4 in., and an air content of 6 +/- 2 percent. AHTD also requires Class S(AE) concrete mixtures have a maximum water to w/cm of 0.44, a minimum total cementitious material content of 611 lb/ yd^3, and a coarse aggregate meeting either the AHTD Standard Gradation or the AASHTO M43 #57 Gradation.

AHTD allows the use of fly ash and slag cement in bridge decks. Fly ash can either be Class C or F, with no mixing of the two. The maximum fly ash replacement rate is 20% by weight, and the maximum slag replacement rate is 25% by weight. If both materials are used, the maximum replacement rate is 20%, by weight, for both materials.

Subsection 802.09 (d) states "When a transverse strike-off is used the rate of placement and consolidation shall be adequate to ensure that no concrete will take its initial set closer than 100 feet behind the strike-off. Compliance with these requirements may require the use of a retarding agent." AHTD allows the contractors to place the whole deck continuously in one operation. If the contractors choose this option, the concrete must remain plastic during the entire length of the pour. Rather than casting the positive moment regions of the bridge deck first followed by the negative moment regions, most contractors are choosing continuous casting, or pours, to avoid the required waiting periods between placement segments.

Placement may be by pump or other method of conveyance as long as the concrete is protected from contamination or segregation. Finishing equipment must shape the concrete to the desired profile and thickness of the finished bridge deck. According to subsection 802.20(a), typically, “the addition of water to the surface to aid in finishing will not be permitted.” When it is allowed, it is only as a fog spray using approved equipment. After finishing, the surface is checked for high and low spots using a 10 foot straightedge in both directions. Deviations greater than 1/8 inch are to be corrected prior to the set of the concrete. Straightedge testing is to be repeated and the bridge deck is to be profiled as soon as the concrete hardens enough to resist damage.

Bridge decks in Arkansas are routinely given a Class 5, Tined Bridge Roadway Surface Finish (802.19(5)), consisting of a burlap drag, followed by tining with a wire rake. The tines produced are to be 1/8 to 3/16 inches in depth, spaced on ½ to ¾ inch centers. The tines run perpendicular to the centerline the full width of the roadway, except the 18 inches nearest the gutter line, which receives a broom finish. Alternately, the surface may be floated with a finned float to produce the transverse grooving of the same dimensions as above. In rare occasions, the bridge may be given a Class 7, Grooved Bridge Roadway Surface Finish (802.19(7)), in which the surface is given the same burlap drag, or a belted finish, prior to curing and then grooved by a mechanical sawing device to produce the same texture.

AHTD specifications allow the use of several different materials for concrete curing. Burlap-polyethylene sheeting, polyethylene sheeting, copolymer/synthetic blanket, membrane curing compounds, and other materials that meet AASHTO M 171 are allowed. AHTD specifications require that the bridge deck have curing compound, meeting AASHTO M 148, Type 1-D or Type 2, applied immediately after finishing at a rate of 1 gallon per 125 square feet. It must then be covered using mats or blankets as a final cure and remain covered for at least 7 days. During these 7 days, the curing materials must be kept "continuously and thoroughly wet" (802.17(b)).

1.5 Research Objectives

The primary objective of the research program is to identify a curing regimen(s) that can successfully reduce plastic shrinkage cracks in bridge decks. By reducing the amount of cracks, the durability of the bridge deck is increased along with the life of the bridge deck. An improvement in durability typically translates into cost savings, as there should be a reduction in maintenance, rehabilitation, and replacement. The data collected in the research program will be used to revise AHTD's current specifications on curing methods for concrete bridge decks. By revising AHTD's specifications, contractors should have better (and possibly fewer) options to choose from when curing bridge decks which will hopefully lead to less cracking in Arkansas' bridge decks.

1.6 Work Plan

To identify a curing regimen(s) that can successfully reduce plastic shrinkage cracks in bridge decks, the research program investigated curing procedures of AHTD and surrounding DOTs and document the curing procedures of 5 Arkansas bridge decks that are being constructed. A laboratory program followed that investigates curing regimens. The work plan was divided into five tasks, as listed and described below.

1.6.1 Task 1: Literature Review

The first task in the research program was a thorough review of relevant literature. The literature review continued throughout the duration of the project, but the major emphasis was concentrated in the beginning of the project. A search of all relevant journal articles, books, and technical reports was conducted. The literature review is provided in Chapter 2.

1.6.2 Task 2: Surveys and Interviews

The second task in the research program was a survey of AHTD Resident Engineers (RE) and/or District Construction Engineers (DCE). The purpose of interviewing these individuals was to determine where most bridge deck cracking occurs within Arkansas and determine the current accepted practices for bridge deck construction on AHTD projects. The REs or DCEs were also asked for their opinion on the potential causes of bridge deck cracking in their area. The second part of Task 2 includes surveying engineers from surrounding DOTs to learn from the experiences and practices of neighboring DOTs. The results of these surveys are summarized in Chapter 3.

1.6.3 Task 3: Bridge Deck Survey

The third task in the research program was a survey of existing bridge decks in Arkansas. The research team examined bridge decks that experienced early-age cracking and some that did not. Selection of decks to be studied came from the RE questionnaire responses. The research team examined the construction practice, the design, the curing practices, and other properties of the bridge decks to determine if there were any common denominators in the cracked or un-cracked bridge decks.

In addition to visual crack mapping, the research team employed new technology, the Digital Highway Data Vehicle (DHDV), developed to measure and map cracks real time. The real time measurement obtained from the DHDV was compared to those performed manually by the research team. The results of Task 3 are further discussed in Chapter 4.

1.6.4 Task 4: Field Study

Task 4 involves a field study of five bridge decks that are under construction in Arkansas. The research team documented and monitored the construction practices and performance of these bridge decks as well as measured the fresh and hardened properties of the bridge deck concrete. Monitoring included documenting everything from ambient conditions when concrete is placed to examination of the new deck for cracking. The results of the Task 4 are presented in Chapter 5.

1.6.5 Task 5: Laboratory Study

The purpose of the laboratory study was to test curing procedures in a small scale environment where variables could be controlled or monitored. In this way the effects of changes to curing practices could be observed. Various curing methods were applied to small slabs comprised of a standard AHTD bridge deck concrete mixture with the coarse aggregate removed. The slabs were subjected to cycles of heated air, light and wind to simulate conditions experienced by a concrete deck constructed in the summer construction season. The resulting cracking was measured and the effects of changes in variables were compared based on those results. Additional batches were made to investigate other characteristics of bridge deck concrete, such as time of setting and bleed rate. This work was all completed in the concrete lab at the Engineering Research Center (ERC), Fayetteville, AR. The results of this laboratory study are discussed in Chapters 6 and 7.

CHAPTER 2

LITERATURE REVIEW

2.1 Factors Affecting Plastic Shrinkage

The roots of plastic shrinkage and the associated cracking branch into all aspects of concrete engineering, including the materials and admixtures, structural design and detailing of the deck, and construction practices used in the field. A review of available research literature was made to qualify and quantify the influences of these components.

2.1.1 Design Considerations

2.1.1.1 **Girder Configuration**. The analytical study completed by Krauss and Rogalla found that stress in the concrete deck and therefore the risk of transverse cracking was higher for steel girders than for concrete (1996). The parameter study was based on developed equations for the stress in the deck and correlated to other research when possible. The study also found that deeper girders caused more restraint, increasing the cracking risk. This suggested to the researchers that longer spans would have more transverse cracking due to the larger girders required. Reducing the girder spacing to reduce the required girder size was only partially successful, as narrow girder spacings also increased restraint and therefore cracking potential. Continuous girders pose more cracking risk than simple spans due to the increase in restraint at the supports. The effects of stay-in-place forms on transverse cracking were the subject of some debate among the sources quoted in the report, with some citing the increased restraint and others disagreeing. The parameter study indicated non-uniform shrinkage resulting from their use could increase the severity of deck cracking (Krauss and Rogalla, 1996).

2.1.1.2 **Deck Thickness and Cover.** A Michigan DOT survey quoted in *NCHRP Synthesis 333* (Russell, 2004) found that depth of cover over the top reinforcing steel is "the most significant factor contributing to the durability of the deck" according to states responding. The survey reported 2.5 in. as the most common value used by states. When deck slabs are exposed to deicing salts and use unprotected reinforcing steel, AASHTO bridge design specifications require a minimum of 2.5 in. of cover. Using less cover in conjunction with epoxy-coated reinforcing steel is permitted (AASHTO, 2002). Current AHTD design practice uses 2.5 in. of cover to the top layer of transverse slab reinforcement for both epoxy-coated and non-epoxy coated reinforcements and places longitudinal shrinkage and temperature reinforcement above this layer. Thicker decks can provide more cover, however the previously mentioned parameter study found that non-uniform shrinkage in thicker decks may increase stresses and cracking (Krauss and Rogalla, 1996).

2.1.1.3 **Reinforcing Steel.** Steel reinforcement can create weakened planes prone to cracking over the top transverse bars. To reduce the effects of this cracking, top and bottom transverse bars should be offset vertically and longitudinal temperature and shrinkage reinforcement should be placed above the transverse bars. A greater number of smaller bars at a narrower spacing are preferred over larger bars at a larger spacing (Rogalla et al., 1995). Purdue University researchers found that current reinforcing codes do not provide enough reinforcement to limit crack development to acceptable levels (Bice et al., 2006). Cracking perpindicular and parallel to reinforcing bars both accelerate the onset of reinforcement corrosion, but cracks parallel to reinforcing bars are more serious because the exposed length of the bar is equal to the length of the crack. Chloride levels have been found to be significantly higher at locations of cracks (Darwin et al., 2006). Even if protected with epoxy-coating, reinforcing steel can experience significant corrosion at discontinuities in the coating (Krauss and Rogalla, 1996).

2.1.2 Material Factors

Optimization of a concrete mixture design can improve both the fresh and hardened properties of concrete. Kansas University researchers recommend a reduction in the paste content of concrete mixtures, as it is the "portion of the mix that ends to shrink and crack" (Darwin et al., 2006). They accomplished this by limiting the cement content to 563 lb/yd^3 and the water to cementitious materials ratio to 0.45. Using an optimized aggregate gradation (known as the Shilstone method) can decrease the paste volume while maintaining workability. More restrictive limits on slump, 1.5 to 3 inches with an absolute maximum of 4 inches, and air content, 8% +/- 1% with absolute limits of 6.5 to 9.5 %, are also recommended. The Kansas researchers have avoided including supplementary cementitious materials (SCMs) because their overall effects on bridge deck concrete are not completely understood (Darwin et al., 2006). Other research has shown that the use of fly ash or slag cement can decrease permeability, but also effects mixture workability and the curing requirements of the mixture (Sanders, 2006).

2.1.3 Construction Practices

Material and design practices specified to minimize or mitigate the effects of plastic shrinkage are wasted if construction practices that affect plastic shrinkage are not considered. Of these, curing practices and weather conditions most directly influence cracking of this nature. However, other variables must also be evaluated.

2.1.3.1 **Placement Size**. Deck placement length has not been found to contribute to plastic shrinkage cracks, but long placements typically require set retardants that lengthen the plastic phase. AHTD construction specifications and bridge plans stipulate that contractors ensure that no concrete takes its initial set before the placement is complete, and suggest that a retarding agent may be required (AHTD, 2003). The longer the concrete remains plastic, the larger the window for plastic shrinkage to occur (Russell, 2004 and Krauss and Rogalla, 1996). Most set retarders also act as water reducing admixtures, reducing the amount of bleed water available for evaporation. This, too, may contribute to plastic shrinkage cracking.

2.1.3.2 **Consolidation and Vibration.** Proper consolidation is an important step in preventing cracks. Insufficient consolidation may leave voids under obstructions that can cause settlement cracks (Krauss and Rogalla, 1996). Vibration may be manual or automated as a part of a finishing system, but it must be timed to “assure close contact with the reinforcing steel after the concrete has ceased to subside” (ACI 345, 2005).

However, the effect of traffic and construction induced vibration on plastic concrete is questionable. As the concrete begins to take initial and final set, construction activity and traffic on adjacent sections can cause the reinforcing bars, formwork, or the entire slab section to move or vibrate. Researchers Issa, Yousif, and Issa found through finite element analysis of the system that "significant deflections were observed… for the portions considered to have wet concrete," resulting in settlement and loss of cover. They also theorized that vibration of the steel reinforcement protruding into the fresh concrete from any source would "cause significant cracking of the concrete at early ages," although analysis of this theory was not included in their research (Issa et al, 2000). Other studies found that well-proportioned concrete can resist damage or bond loss caused by vibration (TRB, 1981) and that no detrimental effects were caused by traffic vibration adjacent to fresh concrete placements (Furr and Fouad, 1982).

2.1.3.3 **Finishing and Texturing.** While conducting research into the influence of mix proportions on plastic shrinkage cracking in thin slabs, Shaeles and Hover noticed parallel cracking due to plastic shrinkage instead of the random pattern they anticipated (1988). Subsequent testing was able to verify that cracks formed "parallel to the straightedge [used as a screed] itself and perpendicular to the direction of travel of the screed." Tests also showed cracking increased with increased screed rate. The authors also contend these lab results are consistent with field observations (Shaeles and Hover, 1988).

The choice of finishing procedures used on bridge decks plays a role in managing plastic shrinkage cracking, as lengthy procedures delay the onset of final curing. Studies reported in Krauss and Rogalla (1996) found that early finishing, double floating, and the elimination of hand finishing reduced the size or number of cracks. The aforementioned

Kansas research study requires contractors casting decks involved in the study to demonstrate that they can successfully mix, place, and finish the decks using actual equipment on test slabs cast on grade. The study suggests that double-roller screeds not be used, recommending vibratory screeds or single roller finishing machines that do not bring as much paste to the surface (Darwin et al., 2006).

Texture is an important part of bridge deck operation, but the method of texturing can impact plastic shrinkage cracking on the deck. Bridge decks in Arkansas are tined, where a metal rake or finned-float is used to produce transverse grooves on the deck. While tining during placement is an effective and economical method to produce texture, its use delays placement of curing blankets, and reduces the effectiveness of membrane curing compounds (Grady, 1984). Grooves sawn into cured concrete previously finished with a burlap or Astroturf drag produce good texture with no significant side effects to the durability of the concrete (Grady, 1984). Grady also surmised costs associated with sawn grooves might be decreased by increased use. More state DOTs are beginning to switch to mechanical grooving over rake tinning, at least on an experimental basis (Xi et al, 2000, Darwin et al., 2006, MoDOT, 2005).

2.1.4 Environmental Factors

2.1.4.1 **Evaporation Rate.** Environmental factors play an important role in controlling plastic shrinkage cracking. The evaporation rate at which the free water on the concrete surface moves into the air is affected by the ambient conditions at the time of casting. Air temperature, humidity, and wind speed determine the ability of the air to take water from the surface. These variables are highly interdependent. ACI 305 (2005) points out that if only one value varies through its expected range, expected evaporations can change by 300 percent. Menzel and Lerch developed equations and nomographs for calculating an expected evaporation rate based on these three variables (Hover, 1992 and ACI 305, 2005). Adopted in the 1960's, the ACI 305 Surface Evaporation Chart has become the industry standard. Uno (1998) developed equations for the direct calculation to calculate the value using easily obtainable weather measurements to simplify the calculations.

However, opinions regarding the nomograph's basis and suitability for current concrete practice vary. Hover (1992) states that widespread publications of this chart have omitted guidelines for gathering input values, such as using average wind speed at the site rather than gust or peak wind speeds from a nearby weather station, which can induce errors up to 100% of the calculated value. Berhane (1992) questions the direct correlation of evaporation from the surface of fresh to concrete to that of an open water interface. Cebeci and Saatci (1992) argue that Menzel and Lerch were dealing with 1960's concrete, with its high w/cm and low strengths. They contend that modern mixes utilizing water-reducing admixtures, fine cements and supplementary cementing materials have low w/cm and do not produce the amount of bleed water produced by older mixes. Others agree that bleed rates vary widely based on mix characteristics (Hover 1992, Cebeci and Saatci 1992, Topcu and Elgun 2004). Thus, the ACI 305 standard maximum permissible evaporation rate of 0.2 lbs/sq. ft./hr may be more than a modern concrete mixture can tolerate.

However, this should not suggest the discontinuation of evaporation rate calculations. It does suggest clear and limited use is required. By measuring and analyzing meteorological data from across the state, Alabama DOT was able to produce "best estimate" evaporation curves used in calculating expected evaporation rates for planning deck placements (Carden and Ramsey, 1999). Onsite personnel can then measure variables on the day of the pour, calculate actual evaporation rates, and adjust placement and curing procedures accordingly. Concrete sensitivity to evaporation is not significantly different than the human body, according to Hover (1992). He suggests that an observant person onsite can sense critical evaporation conditions based on their own comfort. For a more analytical approach, ACI recommends measuring actual evaporation rates based on actual weight of water lost from a shallow pan at the jobsite (ACI 305, 2005).

2.1.4.2 **Concrete Temperature**. Consideration must also be given to concrete temperature and moisture level. High fresh concrete temperature increases water demand and speeds hydration, thereby increasing the need for replacement moisture. State agencies typically limit fresh concrete temperatures to 45°F to 90°F (Russell, 2004). AHTD current specified limits are 50°F to 90°F (AHTD, 2003). Cooling mix water and aggregates can control mix temperatures (Krauss and Rogalla, 1996).

2.1.4.3 **Ambient Conditions**. According to *NCHRP Synthesis 333* (Russell, 2004), transportation agencies set permissible air temperatures at the time of placement to 35°F to 90°F to limit plastic shrinkage and other deleterious effects on concrete. Wind and relative humidity are typically not limited to specific values, although most recommend avoiding windy days. While ambient conditions at the time and location of placement cannot be fully controlled, their effects on concrete can be mitigated. Placements made in early morning hours experience lower temperatures, less solar heat gain, and typically higher relative humidity during placement. However, these values peak in the first 12 hours of the curing process (Krauss and Rogalla, 1996). If concrete is placed in the evening, it can benefit by curing when air temperatures and wind speeds are typically at their lowest (Carden and Ramsey, 1999). If concrete placement at adverse times of the year or day cannot be avoided, procedures should be implemented to help avoid plastic shrinkage, such as wind breaks or shades.

2.2 Curing

Ineffective curing is reported as the most common cause for plastic shrinkage cracking (Russell, 2004). ACI Committee 116 (1967) defines curing as the "maintenance of humidity and temperature of freshly placed concrete during some definite period following placing, casting or finishing to assure satisfactory hydration of the cementitious materials and proper hardening of the concrete". This "maintenance of humidity and temperature" would not be necessary if ambient conditions were such that evaporation did not remove the bleed water from the surface (ACI 308, 2005). However, bridge decks are subject to high evaporation rates, due to casting conditions and their large surface-to-volume ratio. Therefore, effort is required to maintain "satisfactory moisture". There are two major mechanisms for maintaining this "satisfactory moisture content"; wet cures and sealing cures (ACI 308, 2005).

2.2.1 Wet Cures

Wet cures, also known as water cures or moist cures, maintain a constant supply of moisture to the exposed surfaces of fresh concrete. This moisture serves to balance the water lost to evaporation. Wet cures take various forms in bridge deck construction. Water ponding consists of building an impermeable barrier around the perimeter of the slab and filling this reservoir with water. This barrier can be hard to maintain (ACI 308, 2005), and evaporation is still an issue. Fogging, or spraying, is the use of specialty nozzles to produce a mist over the surface of the concrete surface. Fogging can be applied almost immediately, cooling the surface and providing added humidity. Water availability, proper equipment needs, and air temperature are the controlling factors for this system (Krauss and Rogalla, 1996).

Water-saturated mats made of burlap, cotton, or synthetic fibers placed on the surface of the concrete comprise one of the more common forms of moist cure, according to the *NCHRP Synthesis 333* (Russell, 2004). Proper use of mat cures requires the mats be wetted prior to placing, or the surface of the concrete be wet enough to keep the mat from absorbing water from the concrete, exacerbating the problem. Mats must be placed as soon as finishing is complete, but the concrete must be mature enough to support the mats without damage. Finishing, texturing, and testing requirements also delay mat placement. Saturation of the mats must be maintained, as intermittent drying could do more harm than good (Krauss and Rogalla, 1996). 90% of states surveyed by the Michigan DOT require the mats to remain in place for 5 to 14 days (Aktan and Fu, 2003). A Texas DOT study found four days of curing to be adequate for its bridge deck concrete (Garcia et al, 2005), but most states require 7 days (Russell, 2004). AHTD and

AASHTO require 7-day moist cures for bridge decks (AHTD, 2003 and AASHTO, 2002).

2.2.2 Barrier Cures

Barrier cures do not add water to the system. They seal in the moisture already present in the system. Polyethylene or other plastic sheeting, applied to the exposed surface of the deck in a similar manner to mats, acts as a barrier to evaporation. Waterproof materials should meet or exceed AASHTO M171. Special attention must be paid to sealing edges, laps, and tears for the sheeting to be effective. Caveats to the use of clear plastic are increased concrete temperatures (Krauss and Rogalla, 1996) and discoloration or mottling due to wrinkles in the plastic (ACI 308, 2005).

Another type of barrier cure is the chemical curing compound. Chemical curing compounds are sprayed on the surface after finishing is complete and form a membrane that resists evaporation. The curing compound should be applied just as the bleed water disappears from the surface (ACI 345, 2005) to avoid trapping water under the membrane. Application is typically at a rate of 200 sq. ft. per gal. and in two perpendicular passes to insure even coverage. AHTD specifies a minimum rate of 125 sf/gal (AHTD, 2003). White-pigmented compound reflects solar radiation and keeps heat gain to a minimum (ACI 308, 2005). Questions have been raised about the compound’s ability to defend against evaporation. Shariat and Pant (1984) found that moisture loss exceeded the rate allowed by AASHTO M148 when the compound was applied at the specified rate to a surface grooved within FHWA guidelines. They found that the sides and bottom of the grooves did not receive the same coverage as the top surface, thus allowing water to escape. Environmental concerns are also raised about the volatile solvents used in these products, but water-based compounds do exist (Caltrans, 2003).

2.2.3 Combined Curing Systems

States responding to the *NHCRP Synthesis 333* (Russell, 2004) survey described multiple variations of these methods. Most use combinations of these methods to mitigate each method's shortcomings. Fogging the deck as finishing is completed or an application of clear or white curing compound after initial set can help to retain moisture until moist coverings can be applied. Plastic sheeting placed over burlap mats can prevent the mats from drying, reducing rewetting demands on the contractor (Krauss and Rogalla, 1996).

2.2.4 Testing Cure Effectiveness

Current specifications for the curing materials commonly used in bridge deck construction, liquid membrane-forming curing compounds (AASHTO M 148) and sheet materials (AASHTO M 171), base their evaluation of the materials' effectiveness on two separate tests. Curing compounds are tested for effectiveness using AASHTO T-155 (ASTM C156), where mortar samples topped with the compound to be tested are exposed to controlled evaporation rates and then weighed to determine moisture loss. The moisture loss is limited to 0.11 lb/ft^2 in 72 hours. Sheet materials for curing concrete covers plastic and paper sheet goods designed to prevent water from evaporating from the surface of the concrete. It does not cover cotton or burlap cloth used with additional water applied to prevent the evaporation. It does cover the plastic portion of the poly-burlap mats made for curing concrete flatwork, such as decks. The sheet materials are tested for water vapor transmission using ASTM E 96.

ASTM C1151, "Standard Test Method for Evaluating the Effectiveness of Materials for Curing Concrete," was used for sheet and membrane curing materials, prior to its withdrawal in 2000. The test was comparison of the absorptivity of mortar samples cured with the materials to be tested to that of samples air cured and samples sealed with plastic. The testing was done by exposing thin, desiccated slices of the mortar from the top and bottom of the samples to a moisture source and weighing to determine the amount of absorbed water. A less absorptive sample was deemed to be more completely hydrated, thus more effectively cured (ASTM, 2000).

A Belgian researcher suggested a correlation between the measured compressive strength and the relative hydration of concrete samples, related to the effectiveness of methods used to reduce evaporation from the samples. Samples subjected to different curing efforts and moisture loss measurements had a linear relationship between the relative strength and the level of curing effectiveness (Audenaert and DeSchutter, 2002).

Kraai proposed a simple relative test using two thin mortar slabs, 2 foot by 3 foot by 0.75 inches. One slab was used as a control while the other was varied in only one way. After exposure to identical evaporation conditions, the cracking was measured on each slab. The variable changed could then be said to cause more or less cracking. The use of the mortar or paste portion of a concrete was chosen because it was thought that this is where cracks occur. Also, the dimensions of the slab were designed to mimic that of a lager slab, and any coarse aggregate would have changed the scale effect (Kraai, 1985).

2.3 Cracking Measurements

Bridge deck cracking is usually evaluated through visual crack mapping. In a survey of state departments of transportation conducted by researchers at Brigham Young University, all 28 respondents "cited the routine use of visual inspections for bridge deck condition assessments." The responding states reported using the visual inspections to assess "cracking, joint spalling and surface scaling" (Hema and Guthrie, 2005). Traffic is diverted from the survey area, an area is gridded off, and the lengths and location of cracks are measured and recorded. This process is labor-intensive and hazardous (Wang, 2000), as traffic must be routed away from the survey area, but the surveyors are not completely isolated from traffic hazards.

2.3.1 Survey Automation

Attempts have been made to automate this process. In a study to record and quantify bridge engineers' qualitative opinion of "the time to rehabilitate", researchers marked deck distresses with water based paint and photographed them with a 35 mm camera in a series of pictures. The photographs were then digitized and rectified to produce plan view images of the damaged areas. These were then given to engineers experienced in bridge rehabilitation decision-making. By quantifying the engineers' determinations of when to rehabilitate the mapped decks, and applying regression techniques, the researchers attempted to generate equations to recommend repairs based on area of damage. Results from their research show the probable terminal damage level is between 5.8% and 10% of deck area based on local standards, although this was mostly in reference to spalled or delaminated areas (Fitch et al., 1995).

Schmitt and Darwin conducted a detailed survey of 40 Kansas bridge decks in their investigation of “probable causes of cracking in bridge decks” (1999). Their survey methodology was to map the cracks onto prepared scale drawings of the deck using a measuring tapes to locate the cracks and measure their lengths. The maps were then digitized, and a computer program calculated the crack density as a length per unit area of bridge deck. Plotting these densities against data from the construction records of the bridges, they concluded that cracking increased with higher values of slump, compressive strength, water content, and cement content, and decreased significantly with air contents greater than 6% (Schmitt and Darwin, 1999).

2.3.2 Pavement Distress Surveys

According to NCHRP *Synthesis 334*, pavement distress surveys are increasingly completed using automated detection techniques (McGhee, 2004). Most systems in use or in development use a vehicle mounted imaging system to capture pavement images. Pavement distress, such as cracks, are then detected, measured, and catalogued from the images, either manually by an operator or by automated crack detection software. The analyzer software determines cracking by analysis of pixel grayscale variations. Crack size ranges are routinely reported with a 0.08 to 0.125 in. (2 – 3 mm.) minimum visible crack width for both types of systems. Automated analysis software is currently limited by its ability to register small cracks and avoid false determinations caused by rough or tined pavements (McGhee, 2004).

2.3.3 Bridge Deck Survey Results

The results of these surveys, regardless of source, are typically reported as total crack lengths or cracking length per unit area. Pavement cracking is currently evaluated with one of three emerging standards: the AASHTO provisional standard, the World Bank Universal Cracking Index, and the Texas DOT method (Wang et al., 2002). The first two protocols give a measure of the cracking intensity, defined as a length per unit area. The Texas DOT method yields different measures for each type of crack; cracking length per 100 foot of surface for longitudinal cracks and counts of transverse cracks that extend across the full lane, with less than full width cracks counted as a partial crack. The results of all three are a numerical evaluation of the problem spots on a pavement section and the associated severity levels.

No such indices currently exist for bridge deck surfaces. The *AHTD Bridge Inspection Manual* (AHTD, 2005) does indicate that the National Bridge Inspection Standard code for the deck should be reduced as cracking increases. The NBIS rating is from 9 to 1.The AHTD manual states that a concrete bridge deck could be rated 8, or very good, with "minor transverse cracks", or rated 7, "good", with "sealable cracks". A bridge deck is rated 6, "satisfactory", if it has an "excessive number of open cracks (5' spacing)" and 5, or "fair", if cracking accompanies spalled or delaminated areas and section loss. Respondents to the aforementioned Brigham Young University questionnaire reported that deck cracking requires action when cracks "attain moderate width and density, impact greater than 30 percent of the deck area, have widths exceeding 0.0625 in., compromise the structural capacity of the deck, or accompanied by efflorescence or discoloration due to the rusting of the reinforcement." (Hema and

Guthrie, 2005). ACI 201.1R-93 gives visual guides as to the nature of various types of cracking. Post-plastic phase cracks should not have the torn appearance that plastic shrinkage cracking have, but instead have a clean fracture surface (ACI 201, 2005).

2.4 Conclusions

Changes to construction practices along with improvements to materials work to reduce shrinkage cracking in bridge decks. The University of Kansas research team found that it was “possible to develop nearly crack-free bridge decks” by implementing the “best practices”, such as quick and proper curing and not over-finishing (Darwin et al., 2006). These changes are not radical, but do require contractors to understand and agree to the goals of good construction techniques. Missouri DOT’s top recommendation was increased training for and coordination among the engineers and contractors involved in bridge deck construction (MoDOT, 2005). Arkansas’ specifications match well with most of the prescribed practices, although some areas, such as curing application time limits, should be more clearly defined, and the use of mechanical grooving in place of tined surfaces should be investigated. Ongoing study of these issues is also required, as the design methods, materials, and construction techniques change rapidly, sometimes producing effects that are not better for the long-term durability of the bridge deck.

Chapter 3

Engineer Survey

3.1 Conclusions

The goal of the survey was to determine current bridge deck construction practices in the AHTD construction offices and in the surrounding state departments of transportation. A survey was drafted to question AHTD Resident Engineers and District Construction Engineers on the current local methods and materials used in bridge deck construction in their areas of authority. Although all of the Department's thirty-two Resident Engineer offices within the Construction Division's ten districts are governed by the same *Standard Specification for Highway Construction*, variations in local contractors, terrain, climate, geology, histories, and personnel each factor into the actual construction practices of each office. Documenting these variations and the associated experiences with bridge deck cracking should yield indicate as to where early-age bridge deck cracking is most prevalent in the state and identify possible causes and solutions.

A similar survey was to obtain the same information from engineers representing the Department of Transportation (DOT) in each of the surrounding states. Information shared by other states should indicate whether they face the same issues relating to bridge decks as Arkansas, and identify the DOTs' attempts to solve the problems and their success in doing so. Such information might eliminate potential dead-end measures or provide new directions for this research.

3.2 Survey of AHTD Resident Engineers

As previously mentioned, AHTD's construction management is handled through ten Construction and Maintenance Districts. Each district is made up of six to eight counties and has a central office overseen by a District Engineer. A District Construction Engineer (DCE) reports to the District Engineer and coordinates the activities of 3 to 5 satellite Resident Engineer (RE) Offices that directly represent the Department in field construction projects. The RE offices provide onsite administration and construction inspection for construction projects built by Contractors for the Department. The RE is responsible for the coordination of inspection of the Contractors' work and its adherence to the contract documents. The engineers who hold the RE position are experienced field engineers all of whom hold an engineering degree and all of whom are registered Professional Engineers in the State of Arkansas. The Department also requires that REs be certified in materials testing, including concrete testing. Thus, these engineers are experienced in Departmental policy and procedures, current construction practices, and concrete mixture design and testing.

3.2.1 The Survey

The following survey was distributed to all 32 RE offices, 10 DCE, and to the Department of Transportation in each of the six adjacent states beginning in May of 2005. Twenty REs, one DCE, and only two representatives of adjacent state agencies responded. A subsequent slightly modified survey was sent to the remaining REs in September of 2005. Eight additional REs responded to the second survey. Discounting two vacant RE positions, the response rate was 93% of REs.

The survey questions are listed below. The revised questions are noted. The questions are followed by tabulated values of responses, or select illustrative or representative answers. This presentation of responses focuses on the AHTD RE responses to illustrate the current practice of bridge deck construction on AHTD projects. Information on obtaining a complete collection of survey answers can be found in Appendix A.

3.2.2 Questions and Responses

3.2.2.1 Recent Experiences

1. *Are there bridge deck placements in your area completed within the last 5 years that experienced early age deck cracking?*

- Yes – 17 respondents (61% of respondents, 53 % of REs) reported some cracking problems.
- No – 11 respondents (39% of respondents, 34 % of REs) reported no problems of this type.

Please give as much information as possible: project, bridge, time of year, when cracking was discovered. Were the causes investigated, and was there any remediation performed? What were the results of any repairs? Were there further problems with the deck?

Respondents who reporting cracking problems listed between 1 and 10 instances each, with the average number being 2.35 problem decks. The information furnished varies, but trends exist. Most decks were reported as placed during the summer. Cracking was discovered usually after seven days, but some cracks were not seen until final inspection or until the bridge was opened to traffic. Causes were not typically investigated, but were either attributed to a construction or material issue, or staged construction vibrations. Some decks were left as is, others were sealed with a crack sealant, and some decks were top-coated with a polymer or epoxy sealant. No further issues were noted.

2. *Are there bridge deck placements in your area completed within the last five years that did not experience early age cracking, due to routine construction procedures or due to unusual measures taken to improve deck performance?*

- Yes – 17 respondents (61%, 53% of REs) reported some bridge desks without cracking problems.
- No – 7 respondents (25%, 22 % of REs) reported no decks cast without some sort of cracking problem.
- NA – 4 respondents did not provide a definitive answer to the question.

Some respondents did not comment further. Eight of 17 affirmative answerers listed bridges that did not have cracking problems. Some were on the same jobs or by the same contractor as problem bridges from Question 1. Most comments claimed no specific measures were taken beyond those called for in AHTD specifications.

3. *Do you believe that early-age deck cracking is a problem on typical bridges currently constructed in Arkansas?*

- Yes – 12 (43% of respondents, 38% of REs)
- No – 12 (43% of respondents, 38% of REs)
- Don’t Know – 1 (4% of respondents, 3% of REs)
- No Response – 3 (11% of respondents, 9% of REs)

Most respondents responded simply “yes” or “no”. Others placed qualifiers such as continuous pours, proper curing, concrete girders, box-culverts, or staged construction on their responses.

4. *What do you believe are the principal causes of and solutions to such cracking?*

21 respondents (75%) listed possible causes and/or solutions to early age deck cracking, 6 respondents (21 %) offered no answer, and one “did not know.”

Responses varied, but certain key issues were prevalent. Concrete mixture issues, such as high cement or water content and bad retardant doses, tied with poor curing or construction practices for the most mentions at nine. Vibration due to staged construction or construction operations were mentioned six times. Weather conditions were blamed four times and continuous pours received two mentions.

5. *Are there other problems or concerns about the construction of bridge decks, related to design, materials, construction specifications, etc?*

- Yes – 10 respondents (36%), listed a concern.
- No – 11 respondents (39%) said they had no other issues.
- No answer – 7 respondents (25% of responses) gave no answer.

Most of the issues did relate to previous deck cracking concerns, or related problems. Some questioned whether tighter cementitious material controls or curing specifications might decrease the cracking problems. Others suggested that more staged construction or super-elevated bridges might be contributing to the issue. Two questioned they experienced more cracking in concrete girder bridges than typical steel girder bridges. One questioned why there were cracks in the deck at or near joints in the rail after the rails were placed. And finally, one respondent suggested more sawn joints in the slabs to control drying shrinkage cracks.

3.2.2.2 Bridge Deck Construction Practices, Concrete Mix Designs & Materials

6. *When does the Contractor submit concrete mixture designs for bridge decks to be placed on a job – at pre-con meeting, month before placement, day before placement?* REVISED SURVEY: *week before placement? Are most transferred from a previous job?*

- Week before– 2 respondents (7%)
- Month before – 11 respondents (39 %)
- At Pre-construction conference – 4 respondents (14 %)
- Other – 9 respondents (32 %)

Respondents giving other responses wrote things like “well in advance”, “early in the job”, or “all of the above”. Some respondents mentioned that the requirements for approval of trial batches by AHTD forced the contractor to submit early. Others said some contractors needed prodding to get submissions in.   Concrete supplier selections also played a role. Respondents wrote that where there is only one major supplier, the designs change little and are, thus, easy to transfer from job to job. However, according to one respondent, if a contractor had difficulty finding a supplier, then the mix design would be delayed. Early responses of transferring approved mix designs from previous jobs prompted the addition to the revised survey. Including mentions from both, eight respondents (29%) said mixtures were commonly transferred from other bridges.

7. *What types and brands of cement are the Contractors supplying for their Class S(AE) bridge deck concrete mixtures?*

- Type – Only 17 out of 27 responses to this question listed the type in the answer. Of those, 15 (88%) said Type I was the predominate type used, Type I/II received 1 (4%) response, and Type II received 1 (4%) mention.
- Brands – Much more variety exists in the brands mentioned. The results are shown in Figure 3.2.2.2-7. Some respondents included more than one brand.

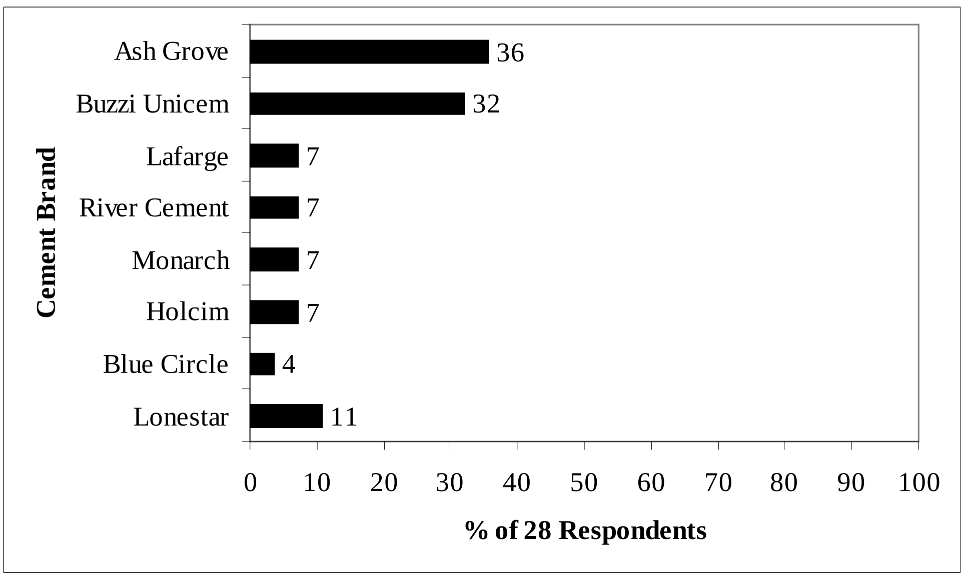

Figure 3.2.2.2-7 Brands of Cement Listed in Survey Responses

8. *Are the Contractors choosing to replace portions of the cement with fly ash or ground granulated blast-furnace slag?*

Fly Ash:

- Yes – 20 respondents (71 %)
- No – 5 respondents (18 %)
- Varies – 3 respondents (11 %)

GGBFS

- Yes – 2 respondents (7 %)
- No – 14 respondents (50 %)
- Varies – 1 respondents (4 %)
- No Response – 11 respondents (39 %)

9. *What admixtures do the Contractors routinely request in bridge deck mixtures; specific air-entrainers, water reducers, set retarding agents?*

Air Entraining Admixtures (AEAs)

- Grace – 10 respondents (36%) said their bridge decks included Grace AEAs.
- Degussa – 5 respondents (18%) said their decks routinely used Master Builders (or Degussa products).
- Both – 3 respondents (11%) said the suppliers in their areas used either Grace or Degussa chemicals.
- Yes – 8 respondents (29%) merely affirmed that their decks used AEA.
- Varies – 1 respondent (4 %) said it depended on the supplier/contractor.

Set-retarding Agents

- Grace – 9 respondents (32%) their bridge decks included Grace products.
- Degussa – 5 respondents (18%) said their decks Degussa products.
- Both – 3 respondents (11%) said the suppliers in their areas used either Grace or Degussa retarders.
- Yes – 8 respondents (29%) affirmed that their decks used retarders.
- No Answer – 2 respondents (7%) did not address set retarders in their answer.
- Varies – 1 respondent (4 %) said it depended on the supplier/contractor.

Water Reducing Admixtures

Not all respondents addressed this in their answers. The following responses are of note: “I haven’t had any requests to add a water reducer to a bridge deck.” “Seldom have I seen a request for a water reducer.” One respondent mentioned that WRDA 79, a Grace product, was used as retarder and a water reducer. Only 6 respondents (21%) included water-reducing admixture in the answer.

10. *What are your criteria for accepting admixtures to concrete mix designs?*

- It’s on AHTD Qualified Products List – 15 respondents (53 %)
- Based on Trial Batch results – 2 respondents (7%)
- Both of the Above – 3 respondents (11%)
- Used per AHTD Specifications – 4 respondents (14 %)
- Used per Manufacturers’ Specifications – 2 respondents (7%)

11. *Do Contractors in your area use ready-mix plants or onsite plants for concrete supplies?*

- Ready-Mix Suppliers – 27 respondents (96 %)
- Onsite Plants – no respondents
- Both – 1 respondent (4 %)

12. *Do bridge deck concrete mixtures in your area use crushed stone or graded gravels?*

- Crushed Stone – 16 respondents (57%)
- Graded Gravels – 6 respondents (21%)
- Both – 6 respondents (21%)

13. *Do these answers vary seasonally or by Contractor?*

- By Season – 1 respondent (4 %)
- By Contactor – 6 respondents (21%)
- Both of the Above – 1 respondent (4 %)
- By Location – 2 respondents (7%)
- No Variation – 15 respondents (53 %)

3.2.2.3 Bridge Deck Placement Planning

14. *When does the Contractor establish schedule of bridge deck placements for a job and make such a schedule known to the RE's representative?*

Responses varied to this question, and some respondents gave multiple or qualified answers.

- Two respondents (7%) simply said, "As soon as possible."
- Three respondents (11%) said that no schedule was given, or that the REs' representative simply knew when placements were eminent by being onsite.
- Three respondents (11%) said that a tentative schedule was usually given early in project, even at the pre-construction conference, but as one respondent said, "We do not put much trust in the pre-con schedule!"
- Three respondents (11%) mentioned weather as a schedule control or possible cause of delays.
- Three respondents (11%) alluded to the common request by contractors to change the pouring sequence in the plans to a continuous pour as source of placement schedule issues.

- Thirteen responses (46%) included an actual range of weeks or in one case days lead time in Contractors' schedules of deck placements. The majority of these ranges fell between 1-3 weeks.
- Finally, two respondents (7%) said, "It varies."

15. *What notice is given to RE's office for dates and times of specific bridge deck placements? Revised: Is there a pre-pour meeting? What is discussed?*

Answers were similar in variety and content to question 14. In fact, four respondents (14%) simply referred to their previous answer.

- Five responses (18%) gave a typical notice of 1-3 days.
- Eleven respondents (39%) said the notice was measured in weeks, but nine of these (32% of total) said it was usually "a week".
- Three respondents (11%) said the REs' inspectors were onsite and could tell when a placement was imminent.
- Four respondents (14 %) to the survey gave no response to this portion of the question.
- One respondent (4%) said it "Varies."

Seven respondents also mentioned some of the causes for the schedule changes. Four listed weather forecasts. One said corrections to deck steel or grades, and one said schedule changes were made based on the Contractors equipment and materials supplies.

Is there a pre-pour meeting?

Based on 2 mentions of these meetings in responses to the original survey, this specific question was added. Of the 8 responses to the revised survey, only 2 (25%) respondents said there is typically a pre-pour meeting, 5 (68 %) respondents said there is not typically a meeting, and 1 (13%) respondent said it depended on the pour situation. As to topics discussed, answers were few, but focused on the Contractor's supplying information such as that listed in Question 16.

16. *What are the Contractor's responsibilities to the Department prior to placing a bridge deck - Supplying grade and thickness tolerances, pour lengths and construction procedures to be used, curing practices and materials, equipment and personnel to be present, etc.?*

Responses included the following items the contractors should submit to AHTD, checks they should make, and other answers regarding deck pour planning. After each are the number of respondents that mentioned that item and the corresponding percentage of 28 respondents.

- All of the above (items listed in the question) – 7 (25 %)
- None of the above – All covered in pre-pour meeting – 1 (4 %)
- Profile grades for steel, deck thickness, steel clearances, etc – 19 (68 %)
- Pour sequence to be used (approved if changed from plans) - 9 (32 %)
- Screed and other construction equipment to be used – 9 (32 %)
- Construction materials to be used (concrete mix, aggregate) – 3 (11 %)
- Proposed curing methods and materials – 11 (39 %)
- Construction personnel/labor to be present for placement – 9 (32%)
- Testing plans and testing personnel to be used on pour day – 3 (11 %)
- An approved concrete mix design – 5 (18 %)
- Proper joint settings made for temperature at time of pour – 1 (4 %)
- Methods for controlling concrete temperature (heating or chilling mix) – 3 (11 %)

- Back-up or contingency plans for pour-day – 2 (7 %)
- Contractor to adhere to plans and specifications, unless approved otherwise - 2 (7 %)

17. *What are the RE's responsibilities and procedures for planning a bridge deck placement on a project?*

Responses included the following items the RE should verify, inspect, discuss with the Contractor, plan or provide for and other answers regarding deck pour planning. After each are the number of respondents that mentioned that item and the corresponding percentage of 28 respondents.

- Checking steel girder grades within tolerances – 10 (36 %)
- Inspecting stay-in-place formwork installation – 4 (14 %)
- Inspecting overhang forms – 2 (7%)
- Verify reinforcing steel placement – 10 (36 %)
- Verify deck thickness at screed dry-run –7 (25 %)
- Verify approved mix-design and aggregate gradations – 3 (11%)
- Verify approved pouring sequence – 1 (4 %)
- Check for adequate equipment, material stockpiles, and plant capacity to meet minimum placement rates and adhere to pouring schedule – 2 (7%)
- Provide testing and sampling requirements to Contractor – 1 (4 %)
- Check weather and advise Contractor of related necessary precautions – 2 (7%)
- Provide adequate personnel for testing and inspection – 5 (18 %)
- Verify adherence to plans and specifications – 4 (14 %)

- Provide verification and inspection of all of the above (answers to prev. ques.) –1 (4 %)
- Discuss the plans with Contractor – 1 (4 %)
- Maintain communications and relations with Contractor – 1 (4 %)
- Keep general overview of job and head-off potential problems – 1 (4 %)

Two respondents correctly noted that the RE technically does not plan bridge deck placements, and four respondents did not answer the question.

3.2.2.4 Concrete Placement & Inspection Procedures

18. *What time of day do deck placements typically start? What governs this decision?*

- 25 respondents (89%) said early morning was the preferred start time.
- 3 respondents (11%) said morning or night pours were possibilities.

All comments about the decision criteria mentioned avoiding the heat of the day. Some noted that winter pours might start later, but others commented that available time to complete the work required was a factor. Some respondents mentioned that availability of concrete played a role, and one said this might be easier on a night pour.

19. *What personnel does the RE have onsite for a typical deck placement? Does this vary based on deck size?*

Some respondents listed personnel by AHTD titles, others by deck placement responsibility, while others gave only a number range of personnel. Tabulating these all numerically, the mean number of AHTD personnel onsite for a bridge deck placement was 3. This agrees with the predominant answers given by AHTD title, which was the job's primary inspector (Field Engineer/Senior Inspector/ Inspector), a helper (Inspector/

Construction Aide/ Construction Helper), and materials tester (Materials Inspector/ certified materials technician). Additional personnel included in answers were additional Inspectors as needed, Project Coordinators, and Construction Materials Inspector, for Independent Assurance Samples (IAS) testing. REs and Assistant REs were also noted to make site visits during pours, but most respondents said these were short visits to check on things without disrupting operations.

By job duties, the average three personnel were described as one person watching the placement, consolidation, and finishing, one person monitoring or performing materials testing and a third person monitoring the concrete at the truck, via the batch tickets.

16 respondents (57 %) said the crew size does vary, either with pour size or personnel availability. 3 respondents (11%) said it does not vary. 9 respondents (32 %) did not address this part of the question.

20. *Are batching operations monitored continuously?*

- No – 20 respondents (71%) said no, or not unless there was a problem
- Yes – 3 respondents (11%) said they had people at the batch plant sometimes, depending on the size of the bridge deck.
- Other – 4 respondents (14%) said that they monitored the batching operation by checking the required computerized printout of batch weights on the concrete truck tickets at the site.

21. *Do the Contractors typically use their personnel or independent materials inspectors for onsite concrete testing? Who does the Department's sampling for verification and acceptance testing?*

Contractors' testing is done by:

- Contractor's own personnel – 1 respondent (4%)
- Independent testing lab personnel – 8 respondents (29 %)
- Combination of these two – 12 respondents (43%)
- Varies – 6 respondents (21%)

Some respondents noted that while many Contractors now have Center for Training of Transportation Professionals (CTTP) certified testing personnel; some would still use an independent laboratory for testing compressive strength cylinders.

Department's testing is done by:

- Construction Materials Inspector – 7 respondents (25 %)*
- A CTTP Certified Inspector – 7 respondents (25%)
- A CTTP Certified person in RE office – 8 respondents (29 %)
- Personnel / Inspector from RE office – 5 respondents (18%)
- No Answer – 3 respondents (11 %)

* Some respondents said they used their Const. Mtls. Inspector, if available, otherwise a certified inspector or helper.

One respondent noted that the District Materials Supervisor was responsible for testing the compressive strength cylinders for the Department.

22. *Who is responsible for monitoring the weather (temperature, wind speed, etc) and making adjustments to planned or ongoing deck placements? What methods do they use: weather radio, onsite equipment, radio, TV, etc?*

Who is responsible?

- Contractor is responsible – 15 respondents (54%)
- RE or Department is responsible – 2 respondents (7%)
- This is a shared responsibility – 11 respondents (39 %)

How do they monitor the weather?

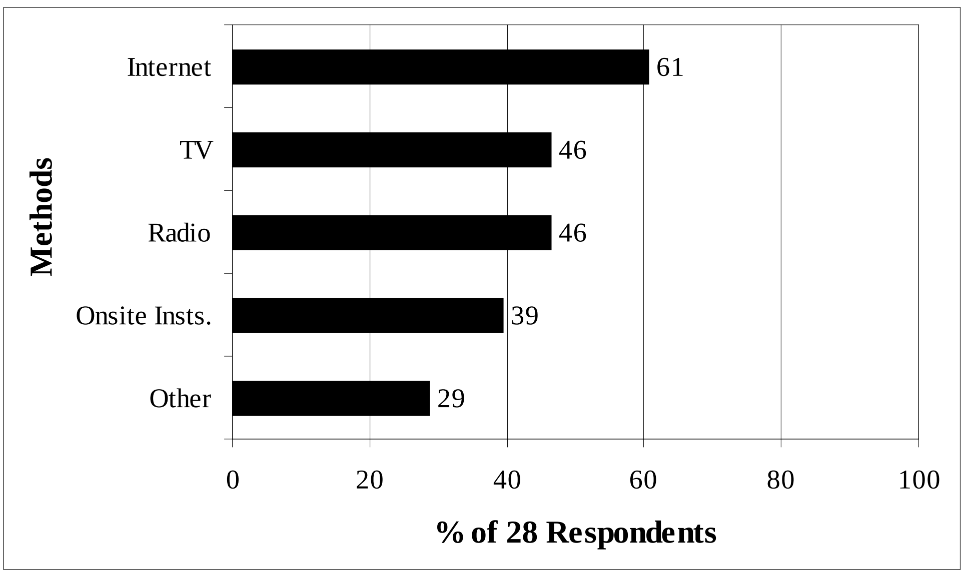

Figure 3.2.2.4-22 Sources of Weather Information Listed in Responses

23. What *documentation does the RE's staff make during deck placements?*

Responses included the following items that are to be documented by the RE's representatives during placement. The numbers represent the number of respondents that mentioned this in their response and the corresponding percentage of the 28 respondents. Some responses included multiple items.

- Soundings of concrete depth/thickness – 20 respondents (71%)

- Concrete cover over reinforcing steel – 19 respondents (68%)
- Results of fresh concrete property tests – 19 respondents (68%)
- AHTD Construction Diary / Bridge Book info – 7 respondents (25%)
- Ambient Weather Conditions (Temp., Wind, etc.) – 6 respondents (21%)
- Concrete temperature – 5 respondents (18%)
- Concrete truck load and unload time – 5 respondents (18%)
- Concrete placement rate – 5 respondents (18%)
- Concrete truck batch / delivery tickets – 4 respondents (14%)
- Unusual events / problems / truck rejections – 3 respondents (11%)
- Approved pouring sequence followed – 1 respondent (4%)
- Straight-edge testing results – 1 respondent (4%)
- Placement start and stop time – 1 respondent (4%)
- Tining / surface finish – 1 respondent (4%)
- Curing application – 1 respondent (4%)

One respondent mentioned a Bridge Deck Placement Form that is completed by the inspector that documents many of these items. A copy of this form can be found in Appendix A.

24. *What documentation does the Contractor make and submit to the RE regarding deck placements?*

Responses included the following items that are to be documented by the Contractor's representatives during or after placement and then submitted to the RE. The numbers represent the number of respondents that mentioned this in their response and the corresponding percentage of the 28 respondents. Some responses included multiple items.

- Concrete fresh property and compressive cylinders tests – 16 (57 %)
- No information required after placement – 4 (14%)
- Copies of concrete delivery tickets – 2 (7 %)
- Deck grades – 2 (7 %)
- Survey notes (if requested) – 1 (4%)
- Same information as RE documents – 1 (4%)

7 respondents (25%) included items of information submitted prior to placement such as mix design, screed type, and planned pouring sequences.

3.2.2.5 Finishing & Curing Practices

25. *What methods are used to control concrete surface evaporation prior to final curing? Clear curing compound, fog machine, or misting sprays.*

- Curing Compound – 27 respondents (96%)
- Misting Sprays – 8 respondents (29%)
- Other – 2 respondents (7 %) – Both responses included wet burlap or burlene mats used for final curing.

26. Are *straight-edging and profiling easily accomplished within the prescribed limits?*

Three responses (11%) offered no opinion on, only descriptions or observations of the surface tests:

- "Straight edging is performed right behind the screed, but profiling is performed after the deck is cured."

- “Straight edging is accomplished during the pour sequence so as to have time for corrective measures.”
- “The[re] is not a clearly defined timeframe in our specs for the straight-edging of the fresh struck-off concrete, except that it will be done after finishing. The initial surface test is required after the deck can be walked on, which is typically the next morning.”

Eighteen respondents (64%) said that it was easily accomplished, or at least possible to accomplish. Many simply said “yes” or “usually”, but others qualified their agreement some:

- “Straight-edging: Yes. Profiling: Not always. Typically the Contractor still has the burlap mat, anchored with forms, tools, etc., over the deck when profiling should be done, and does not have a crew available to remove them.”
- “Not easily accomplished, but it is possible. A 10-foot half moon straight-edge is pulled over the entire deck [transversely,] overlapping each time.”

Seven respondents (25 %) said it was not easily accomplished: Again, some commented as to why:

- “No. The Contractor usually has burlap covering the deck and a lot of equipment scattered about.”
- “The longer the span or placement is the harder it is to meet the straight edge requirement. The initial surface test conflicts with the contractors curing process.”
- “Sometimes it is a struggle to get the contractor to meet requirements.”

27. *Are there issues with tining the surface?*

19 respondents (68%) wrote yes, the tined surface finish has issues. Their comments included discussion of differences between concrete mixtures and inconsistent concrete set, difficulties in getting good, uniform tine dimensions, and the delays tining causes to completion of the curing process.

- "Yes. The inspector has to know when to tine the surface and hold them to it. Too early and the tines will not keep specified dimensions, too late and they'll drag and pull up rocks."
- "Depending on the skill of the contractor's finisher, the tining can be difficult or easy. Catching the concrete surface at the right time for a good tine surface is an art sometime more than a skill."
- "The only issue with tining is that by the time the deck sets enough to tine we are too late applying the curing compound."

9 respondents (32%) wrote they had not experienced issues with the tined surface finish. One respondent summed it up when he wrote, "Depending on the skill of the contractor's finisher, the tining can be difficult or easy. Catching the concrete surface at the right time for a good tine surface is an art sometime more than a skill."

28. *How much time typically elapses between initial placement and final curing placement?*

- Twelve respondents (43%) gave their answer in hours, or a range of hours, the most common of which was 4-6 hours. This corresponded to 5, the mean value of the numerical answers.

- Ten respondents (36%) wrote that the time varied. They attributed the variation to differences in concrete mixture, admixture, ambient conditions, and pour length.
- Four respondents (14%) wrote that the process was completed the "same day" or within "one day".

Two respondents (7%) wrote that final curing was placed as soon as possible.

29. *Is this process complicated by the use of set-retarding agents on continuous pours?*

Fifteen respondents (54%) indicated that they did not see set-retarders complicating finishing and curing operations.

- "Without set-retarding agents, continuous pours are not possible. If agents are added to the mix properly and reduced when you are nearing the end of the pour, there are usually not any problems."
- "I think it would be more complicated without them."

Eleven respondents (39%) disagreed, writing that retarders do complicate the process. They cited the delay caused by the retarders use, but some agreed that they were necessary on large pours.

- "The retarder slows down the tining process and therefore slows down the application of the curing compound."
- "Sometimes, but it is certainly more desirable than the complications caused by not using set-retarding agents. Reducing the retarding admixture as the pour progresses usually works well, but not always, of course."

30. *What materials are used for final curing: burlap, plastic, curing-compound, or combinations?*

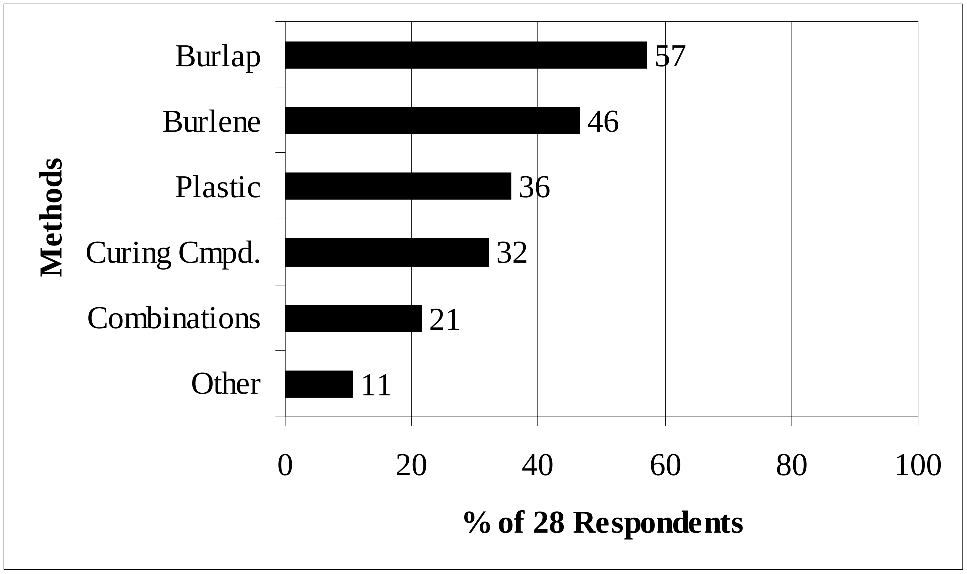

Figure 3.2.2.5-30 Curing Materials Listed in Survey Responses

31. *How is wet curing maintained and for how long?*

Some respondents wrote simply that the curing blankets were soaked with water as needed for duration of curing period. Others described the process with more particulars, some good examples and some not as good.

- "Contractor sometimes will set up a series of soaking hoses supplied from a water tank and pump to keep the surface wet for a minimum of 7 days. Other times, periodic soaking with a water hose is done. Obviously, the first method is most desirable."
- "Wet curing is for a seven-day period, with the Contractor maintaining a relatively constant flow of water across the bridge deck below the burlene covering. Water is typically pumped from a nearby water source (creek or stream) or from a holding tank if a natural source is unavailable."

- "Contractor places polyethylene burlap on deck as soon as it has set sufficiently to walk on without leaving marks and thoroughly soaks burlap with water. Contractor soaks burlap with water several times a day as needed for 7 days."
- "Burlap mats were pulled back and concrete wet down each morning for five days."

As for the duration of wet curing, 23 respondents (82%) wrote 7 days, 2 (7%) wrote 5 days, 1 (4%) wrote "by spec", and 2 (7%) did not respond to the question.

32. *Have you used the Class 7 grooved surface for a bridge deck? How did you perceive the process?*

Yes – 5 (18%) respondents had some experience with Class 7, or mechanically-grooved surface; 4 as repair for tines after surface grinding, one in another state prior to working for AHTD.

No – (82%) respondents said they had not dealt with this process.

Two respondents with experience preferred grooving to tines. Another said that it was "probably better" and had a "more uniform appearance" than tining. One respondent with no experience felt that it is a "good idea" to groove after curing, so that curing would not be delayed until the concrete was set enough for tining.

3.2.3 Survey Conclusions

The survey of the REs does document the bridge deck construction practices on AHTD projects. The spectrum of materials, methods, and results is represented. Variations exist, as shown above, due to all the factors anticipated by the researcher during the development of the survey; local contractors, terrain, climate, geology, histories, and personnel. No single variable, or definable combination of variables, predicts the likelihood of a respondent experiencing a cracking problem in his area of authority.

No factors are purely linked to geographic location. In fact, some factors which one might think would be geographical are increasingly variable. Concrete materials, namely cement and aggregates have traditionally been a local resource. This is still the case in some regions, but others may be seeing the effects of consolidation and market demand on the diversity of cement and concrete suppliers. Some regions have new and different options, while others have fewer sources. Aggregate sources are facing increased quality control and environmental regulation, favoring larger specialized operations over what was easily available locally. Larger workloads in Arkansas in recent years have also brought changes in the contractors working on bridges in a particular area, in some cases changing the predominate way of doing things in such areas.

Responses do not indicate a cracking problem that is isolated to any particular region of the state, either. Only District 3, in the southwest corner of the state, was consistent in their responses, with all three REs reporting problem bridges and believing that cracking was a problem. Continuation of this research by other researchers includes monitoring bridge deck placements in District 3 that include new special provisions regarding curing and concrete mixtures. All other districts varied in their responses within the district. Recent major relocations among Construction Division personnel due to an ongoing AHTD retirement incentive may contribute to the geographic diversity of thought on this issue. Current REs may not have been in their locations long enough for local patterns to exist. In fact, some of the REs responding to the survey have changed locations since responding.

Responses do indicate variations exist in the REs experience with and perception of the bridge deck cracking problem on AHTD. Some REs have not had a problem deck recently, if ever, while others have not had one without a problem. Personal perception of the “cracking problem” may influence this more than evidence presented in the survey. Seventeen respondents out of 28 (61%) reported bridge decks that had early age cracking problems of some type in response to question 1. Of these, only nine (32 % of total) responded to question 3 that they “believe that early-age deck cracking is a problem on typical bridges currently constructed in Arkansas.” The average number of problem bridges reported was 2.35, but with no indication as to the number of other decks cast in the same time period. Twelve of the 17 REs listing problem decks also wrote that they had overseen decks cast that did not exhibit a cracking problem, although not all gave a count of such bridges. It appears that these reported decks that had cracking problems either do not represent the majority of decks cast in the state, or the REs do not see them as a cracking problem. Overall, the responses to question 3, regarding the belief that cracking is a problem in the state, are split 12 to 12.

3.3 Surrounding State DOT Responses

Only two of the six surrounding state departments of transportation, Oklahoma DOT and Missouri DOT, responded to the survey. The Missouri representative also supplied a copy of the report from their joint study with the Federal Highway Administration of the bridge deck construction process in Missouri (MoDOT, 2005) and a copy of the pre-placement training video that resulted from the study. Missouri DOT and contactors representatives watch the video during the pre-placement meeting and discuss issues regarding the planned deck placement.

A partial selection of questions and responses from these surveys are included below. Each response is identified by state abbreviation of the respondent. Information on obtaining the full texts of the questions and responses from these participating agencies can be found in Appendix A.

Questions and Responses

1. *Are there bridge deck placements in your state completed within the last 5 years that experienced early age deck cracking? Please give as much information as possible: project ID, bridge, time of year cast, when cracking was discovered. Were the causes investigated, and was there any remediation performed? What were the results of any repairs? Were there further problems with the deck?*

MO: MoDOT has had several bridges exhibit early age cracking. They have occurred at different times of the year and under various weather conditions. Most cracking is noticed within the first few weeks after placement. Dependent on the crack size they are sealed with pav-on or methacrylate. Each of these products will need to be reapplied in the future. There are no further problems with the decks. However, the bridge condition rating went from a 9 to a 7 instantly**.**

OK: Yes, we have experienced early age deck cracking - US-59 over the Arkansas River in Sequoyah Co. and US-69 over the Verdigris River in Muskogee Co. We plan to flood coat the deck to seal the cracks.

2. *Are there bridge deck placements in your area completed within the last five years that did not experience early age cracking, due to routine construction procedures or due to unusual measures taken to improve deck performance?*

MO: MoODT has had some bridges with little early age cracking. These have been poured in cooler temperatures with proper curing.

OK: Don't know

3. *Do you believe that early-age deck cracking is a problem on typical bridges currently constructed in your state?*

MO: Yes

OK: Yes

4. *What do you believe are the principal causes of and solutions to such cracking?*

MO: Improper curing and placement in high wind areas have been the biggest culprits.

OK: It is my opinion that the cracking is mostly from concrete shrinkage. We could reduce this by better curing practices, eliminating gap graded aggregates, reducing cement content, and better inspection procedures.

5. *Are there other problems or concerns about the construction of bridge decks, related to design, materials, construction specifications, etc?*

MO: MoDOT is currently field testing a new mix design with greatly reduced cementitious material.

OK: Concern: The empirical design appears to be causing longitudinal cracks instead of transverse cracks.

6. *Has your Department performed or requested research related to this issue? Are research reports available?*

MO: Yes. We are tracking deck cracking with our old vs. new curing specifications. We have also performed a joint task force study with FHWA.

OK: No.

7. Is your Department participating in the Pooled-Fund Study at the University of Kansas?

MO: Yes.

OK: Yes.

Both responses indicate that the respective DOTs are facing similar problems to AHTD when it comes to bridge deck cracking. The states have differing levels of specifications, but their responses were not much different than the responses given by AHTD engineers. Missouri appears to be very proactive in their concern about bridge deck cracking. Their answers to the problem utilize improved specification and clearer understanding and implementation of those specifications, followed up with ongoing research to track successful and unsuccessful changes.

3.4 Conclusions and Recommendations

The goal of the Survey was to document the current bridge deck construction practices and materials, in both AHTD Construction Districts and neighboring state DOTs. The information gathered here details not only how these organizations specify and manage bridge deck construction, but also how they perceive bridge deck cracking as an issue. Some of the respondents are aware of the problem, but do not know how to address it. Some are taking purposeful steps to tackle the issue. Some engineers responding to the survey do not see it as a problem, whether due to use of good practices that negate the risk or not.

For Arkansas, tighter guidelines for interpreting the standard specifications for bridge deck construction would help AHTD to produce a more consistent performance of its new bridge decks. Ongoing issues could then be addressed with targeted changes that would be easy to monitor for success. The concept of a pre-placement meeting should be fully instituted as well as standardizing the format and content of such discussion as applicable. Contractors and suppliers should have a clear and consistent understanding of the requirements for bridge deck construction for AHTD.

The survey used in this task should be repeated periodically to track the application of these changes. Slight improvements to the phrasing of some questions and format of some responses would improve the clarity of the answers. However, care should be exercised in constraining possible answers, so that the respondents are allowed to give an honest, thoughtful answer, not just choosing the perceived “correct response” from a group of choices.

Chapter 4

Bridge Deck Survey

4.1 Purpose

This section of the research program consisted of a survey of existing bridge decks in Arkansas. The research team was to examine bridge decks throughout Arkansas in various conditions (good, average, and poor). The research team was then to examine the construction practice, the design, the curing practices, and other properties of the bridge decks to determine if there are any common denominators in the good or poor bridge decks. Bridge decks were to be surveyed that range in age, traffic loads, and type of superstructure, although, emphasis was placed on those bridges identified during questionnaire/survey as good or bad performers.

In addition to manual crack mapping, the research team attempted to employ new technology developed to measure and map cracks in real time. The Digital Highway Data Vehicle (DHDV) was developed at the University of Arkansas and is used by WayLink Systems Corporation to map pavement distress. Data collection is done at highway speed, or near highway speed, depending on resolution, and longitudinal distance. The real time measurement obtained from the DHDV was to be compared to those performed manually by the research team. As discussed in the following section, changes necessitated that a partially-automated method be used to map the cracking of an existing highway bridge. This was compared to the cracking measured by traditional manual crack mapping. The uses and limitations of the systems are discussed.

4.2 Automated Crack Mapping

Developed at the University of Arkansas, the Digital Highway Data Vehicle (DHDV) is an automated system for real-time analysis of pavement surface cracking and other surface distresses (Wang and Gong, 2005). Utilizing a van chassis fitted with a bank of high intensity strobe lights, a fully digital pavement imaging system, dual processing CPU, and a GPS receiver to provide location data, the DHDV is able to record and analyze pavement cracking at 60 mph (97 kph). The digital line scan imaging system currently employed for the DHDV has a transverse resolution of 4096 pixels. Using a 20 mm lens, this produces an image frame of 13.12 ft. by 6.56 ft (4 m x 2 m) with a theoretical visible crack width of 0.04 in. (1 mm). The analyzer software then removes any non-distress noise, connects and enhances images, and applies algorithms to determine the geometric properties of the cracks, such as length, width, and orientation. The software can then store these properties in a distress database and report the type and quantity of cracking for the area surveyed (Wang and Gong, 2005). With other devices onboard for rutting and ride measurements used for pavement management and two digital right-of-way cameras, the DHDV can catalogue miles of roadway conditions and transportation infrastructure while driving down the road.

For the purpose of this study, the lens was changed to 28 mm, yielding an image size of 8.53 feet by 4.27 feet (2.6 m by 1.3 m), and a minimum visible crack width of approximately 0.02 in. (0.5 mm). The DHDV, through a contract with WayLink Systems Corporation, was used to collect and analyze images of the subject bridge decks. As an initial sample to test the methodologies, nine bridges in the northeast corner of the state were chosen from the list of bridges referenced in the surveys, good and bad performers, to be mapped. Their close proximity to each other made for expedient surveying. One additional bridge listed as having early-age cracking in the survey responses was included for its proximity to the researcher's location. This would allow for convenient inspection and a manual survey of the deck for comparisons between the methods.

4.3 Automated Bridge Deck Surveys

The automated surveys were conducted in the afternoon and evening of July 19, 2005 and the morning of July 20, 2005 and were completed using the DHDV and an AHTD van with an amber warning light-bar, used as an escort vehicle. Weather conditions varied from cloudy with intermittent rain storms to bright and clear the following morning. The survey process was to make successive passes across the bridge to cover as much of the whole width as possible. The lighting equipment at the sides of the van did not allow the DHDV to get very close to the concrete rails on the bridge. The survey passes began on the outside shoulders of the bridge, and proceeded inward with each pass, overlapping passes as little as possible. Methods such as flagging, string lines, and pavement markings were considered in an attempt to more precisely align successive passes, but in the interest of minimal site time and traffic disruption, it was decided that the operator of the DHDV could estimate the pass width and align the next pass accordingly. The vehicle speed was limited to approximately 40 mph (64.47 kph). As shown in Table 4.3.1, the bridges were surveyed with 3 or less passes per lane with overlap and in an average time of 17 minutes. The Illinois River bridge on U.S. highway 62 was surveyed by the WayLink crew using the DHDV without an escort vehicle on the evening of July 23, 2005.

Table 4.3.1 Bridges Surveyed with the DHDV

AHTD Bridge Num.	Width (ft.)	Length (ft.)	Lanes On Bridge	Number of Passes	Time to Survey (min.)
06859	75	152	5	10	25
06890	40	106.5	2	5	6.5
06891	40	74.2	2	5	6.5
06772	64	112.2	5	7	20
06912	30	248	2	4	17
06548	80	382.2	4.5	10	31
06544	70	1081.5	4.5	9	20
A6542	40	492.2	2	5	17
B6542	40	492.2	3	6	25
06732	70	381	4	7	20

Although initial plans were based on the use the automated distress analyzer software onboard the DHDV to produce all bridge deck crack maps, the current software is designed to work on standard 4-meter wide images, designed for pavement management system work. The choice to run a different lens to produce finer images changed the image dimensions, rendering it impossible to analyze without costly and time consuming reprogramming that was beyond the scope of the original project. Future refinements to the software may allow for analysis of the collected images for cracking.

4.4 Semi-Automatic Bridge Deck Mapping

In order to preserve the value of the images collected to the original purpose of the research, a semi-automatic process was developed by the researchers. The images would be imported into computer-aided drafting software, the cracking would be digitized manually, and the resulting lengths and widths would be tabulated for the sample area. AutoCAD 2005 (Autodesk) was chosen because the researchers had ready access to it and experience with it. The images would be imported using the IMAGE command, aligned into a composite map for the entire bridge deck, and analyzed for cracking.

The first bridge chosen for analysis was the Illinois River bridge on U.S. Highway 62 near Prairie Grove, Arkansas. The bridge is 70 feet wide and 381 feet in length. The survey of the bridge deck consisted of 6 passes of the DHDV. Each pass of the DHDV generates its own directory of data and image files. These image files are JPEG format pictures numbered consecutively. Scanning through them with a general photo viewer, the researchers were able to eliminate those not on the bridge deck. The consecutive images generated by the line scan camera are by their nature matched to each other at their top and bottom edges. Transversely, however the overlaps vary significantly. Also, the heading of the DHDV varied slightly, so overlaps were not constant for a complete series of images. Thus, some trimming and aligning was required to produce a composite map. This process was completed for a representative 100 feet of traffic lane to match the manual survey sample size. This process took approximately 15 hours for the first sample area, however, this includes development of the process and organization of the image files. The computer used was a Pentium 4 (2.2 GHz) with 1.5 gigabytes of RAM. The individual image size was approximately 260 – 290 kilobytes, but the composite map of the entire bridge deck required 560 images to be combined in one file. This reinforced the decision to limit the mapped area to a representative sample.

The images were manually scanned for any visible cracks. A polyline was drawn over the crack to set up a baseline. Crack widths were measured directly in Autocad by picking the sides of the crack in various locations and allowing the software to measure the distance between points. This direct measurement method was verified by measuring the known line widths on an image of the crack comparator card taken with by DHDV. A Visual Lisp program was written to calculate the length and area for the cracks. The function asked the user to label the crack and specify the average width of the crack. Then the function took the perimeter variable, a system variable stored as the last known perimeter calculated, and saved it as the length. Then the area of the crack was calculated with the known length and width. The length and area were then written to an output file that could be continuously appended and also opened with Excel (Microsoft). From this Excel file, the researchers obtained the total length of cracking in the sample area in feet, the average width of the cracking in inches and the length of cracking per unit area, feet per square foot. Comments on the direction of cracking and the nature of cracks can also be recorded with the crack number for future reference.

4.5 Manual Crack Mapping

The manual mapping for the Illinois River bridge was completed on the afternoon of February 15, 2006. The weather was clear and windy and the air temperature was approximately 60˚F. The process used was similar to that used by AHTD Research Section personnel in a previous bridge deck study (LeClair, 1998). The first step in the mapping process is typically an initial examination of the entire bridge deck to locate the areas most affected by cracking. This is then the survey area. It is generally limited to 100 feet in length unless the additional length would better represent the distress level of the deck as a whole. For the purpose of comparing the semi-automated and manual methods, the same survey area was chosen; the first 100 feet of the eastbound 12 foot driving lane. This section of highway is traveled by over 12,000 vehicles per day, according to the AHTD website. Although the constructed bridge has a roadway width of 70 feet, only one lane in each direction is used by traffic. Traffic control was provided by two AHTD personnel using a simple flagged lane closure with traffic cones along the centerline to keep motorists out of the survey area.

The actual mapping of distress in the survey section began with team members laying out a 100 foot tape measure along the lane edge. This provided longitudinal stationing for the map. A 25 foot tape measure was used to measure transversely from the lane edges. Cracks were then visually located and documented. Some mapping procedures suggest spraying the cracks with water from a small, garden-type sprayer. The water collected in the crack will make the crack more visible as the water on the adjacent surface evaporates (LeClair, 1998). However, the area surveyed was heavily coated with oil and rubber residue, so darker areas would not make the cracks stand out.

The length and location of the cracks were measured and recorded, along with orientation and approximate widths of the cracks on a prepared form with a grid representing the survey section. The widths of the cracks was measured with a crack comparator card which is a clear plastic card marked with thin lines of various widths ranging from 0.002 inches up to 0.125 inches. Although these may be commercially available, the ones used on this project were created using AutoCAD (Autodesk) and printed onto transparency film. See Figure 4.5.1 below.

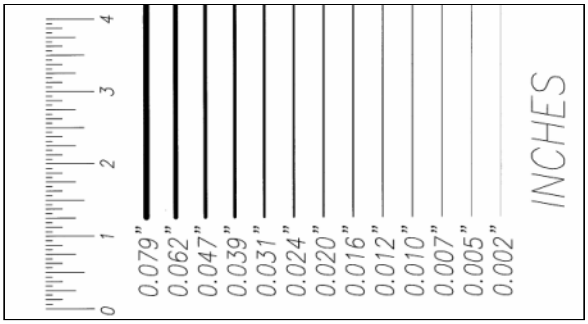

Figure 4.5.1 A Crack Comparator Card used by the Researchers

The data were compiled, totaling up the total crack length for the survey section, calculating a length of cracking per square area (ft. / sq. ft.) of the survey section, and approximating the weighted average width of crack for the survey area. Also, digital photos were taken to document the overall condition of the deck at the time of survey.

4.6 Results

4.6.1 Semi-Automated Bridge Deck Mapping

The twelve foot - seven inch wide by one hundred foot long sample area of the Illinois River Bridge mapped in AutoCAD was found to have 194.5 feet of cracks with a width range of 0.032 inches to 0.174 inches. The average width was 0.089 inches, large enough to be of concern to the durability of the deck. They did not impact a large area of the deck, with only 0.154 feet of cracking per square foot of deck surface. The total area of the cracks was 1.44 square feet, or 0.12% of the surveyed area. The results for the semi-automated mapping of the sample are tabulated in Table 4.6.1.

Table 4.6.1 Cracking Data for Sample Section - Semi-Automated Mapping

Crack ID	Length (ft)	Width (in)	Direction	Crack ID	Length (ft)	Width (in)	Direction
1	20.00	0.1092	long.	11	2.05	0.1283	trans.
2	2.95	0.0337	long.	12	3.15	0.1738	trans.
3	0.69	0.0332	long.	13	4.01	0.1441	trans.
4	31.48	0.0852	long.	14	2.30	0.0974	long.
5	3.90	0.0960	long.	15	1.37	0.0974	long.
6	15.96	0.0792	long.	16	2.78	0.0971	long.
7	44.28	0.0972	long.	17	0.33	0.0325	long.
8	46.24	0.0900	long.	18	1.25	0.0336	long.
9	1.93	0.0673	trans.	19	3.30	0.0972	long.
10	3.24	0.0923	trans.	20	3.30	0.0972	long.

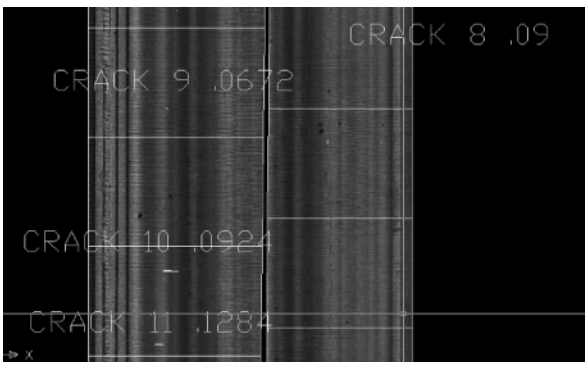

Figure 4.5.2 Partial Crack Map from Illinois River Bridge

4.6.2 Manual Mapping

Manual mapping yielded 416 feet of cracking in the same sample area, excluding cracks 4, 11, and 25. All cracks widths were recorded as less than 0.005 inches. This gives a crack intensity for the sample area of 0.346 linear feet of cracking per square foot, but only 0.173 square feet of crack area, or 0.015% of the sampled surface area. Crack data are reported in Table 4.6.2 below. The Crack IDs used in manual mapping are not the same numbers as the IDs used in the semi-automated process. Cracks 4, 11, and 25 represent areas of short, random map cracking. Including this area in the calculations above, the length does not change, as lengths were not recorded in these areas, but the affected area changes to 70.17 square feet, or 5.9 % of the sampled area.

Table 4.6.2 Cracking Data for Sample Section - Manual Mapping

Crack ID	Length (ft)	Width (in)	Direction	Crack ID	Length (ft)	Width (in)	Direction
1	20	0.005	long.	14	6	0.005	trans.
2	100	0.005	long.	15	6	0.005	trans.
3	100	0.005	long.	16	4	0.005	long.
4	10	36	Map Cr.	17	8	0.005	trans.
5	30	0.005	long.	18	6	0.005	trans.
6	6	0.005	trans.	19	10	0.005	long.
7	5	0.005	trans.	20	12	0.005	trans.
8	7	0.005	trans.	21	8	0.005	trans.
9	7	0.005	trans.	22	4	0.005	long.
10	8	0.005	trans.	23	9	0.005	trans.
11	5	24	Map Cr.	24	8	0.005	trans.
12	9	0.005	trans.	25	2	24	Map Cr.
13	8	0.005	long.	26	6	0.005	trans.
13a	8	0.005	long.	27	4	0.005	trans.

Although the cracks manually documented include the ones recognized in digitized maps, the average widths recorded are smaller, because on close inspection the actual crack openings are not as wide as measured in the computer. The surface opening is quite large, but the root of the cracks is very narrow. It appears the initial crack edges have been worn back by traffic and weathering. This eroded area has a depth of less than 1/8 inch, and does not represent the active crack width.

The majority of cracking was longitudinal. Two long, although not fully continuous, cracks parallel the outside wheel path at 3 feet and 4 feet from the lane edge. A similar crack ran along the inside wheel path at approximately 1 to 2 feet from the centerline. Shorter, intermittent transverse cracks that did not fully cross the lane are also present. Further investigation is required to determine the location of the longitudinal cracks relative to girders or other structural components, and the impact of their location on the deck.

4.7 Conclusions

The differences in the crack widths measured in AutoCAD versus manually were significant, and the amount of cracking measured manually is more than twice that recorded from the digital photos. Various reasons for the differences are possible. The additional transverse cracking may have been obscured in the photos by the tined surface.

Differences in temperature between surveys with the DHDV in July and manual surveys in February would suggest that less cracking might be visible during the manual survey, as the cold temperatures might close many cracks. The time elapsed between surveys makes this difficult to resolve. However, the temperature differential could explain some of the discrepancies in crack width between the two surveys. Visual inspection of types of cracking and measurements of air and surface temperatures as part of future DHDV surveys could improve the accuracy of the subsequent crack maps.

As previously stated, the compositing of the images required approximately 15 hours. The actual crack mapping, however, only required an hour of operator time. The compositing and analysis were both accomplished in an office, out of the elements and hazards of manual field studies. The operation only required one operator. So, including the survey time in the DHDV, (2 men at 20 minutes each) this portion of the deck was mapped and analyzed in under seventeen man hours. The conventional field mapping utilized 2 flagman and 5 researchers for 2 hours in the field and a single man hour for data reduction. So, no reduction in time was seen in this test. However, subsequent computer mapping can be made more efficient and as improvements are made to the automated system, analysis of the entire collection of surveyed decks may be more feasible. Representing over two hundred thousand square feet of bridge deck mapped in 2 eight-hour working days by three personnel for a rate of 4224 square foot / man / hour, this would represent a vast improvement over the current system.

Field work will not continue in this task, however. The sensitivity to capture fine cracks and the ability to process them automatically is not easily attainable, and the development of these capabilities is beyond the scope of this research. Field experience gained as a part of this research has also shown that adequate construction records to determine the likely cause of measured cracking are not available for most bridges constructed. However, WayLink Systems Corporation continues to improve the DHDV and the Pavement Distress Analyzer software. This technology holds much promise in safe, cost effective bridge deck distress mapping for future research or bridge inventory management.

Chapter 5

Field Study

5.1 Introduction

Five bridge decks under construction in Arkansas were observed and the construction practices were monitored and documented. Monitoring included everything from recording ambient conditions when concrete was placed to examination of the new deck for cracking. By documenting the materials and methods used to construct the decks, the conditions under which they were constructed, and their subsequent performance in terms of cracking, factors which increase or decrease cracking should be identified. The field study also included measuring the fresh and hardened properties of concrete sampled from each of the new bridge decks.

5.2 Research Plan

5.2.1 Monitoring and Sampling

The bridge deck placement monitoring consisted of direct observation by research team members of the placement activities, measurement of weather conditions, sampling of concrete used, and measuring cracking in the finished decks.

Construction operations were documented with notes and digital photos to capture the procedures used and the timing of those procedures. Of interest were the methods of placement and initial finishing, concrete placement rate, timing and method of final finishing, curing compound and final curing application. Any problems or delays known to the researchers were also documented.

Weather conditions were monitored onsite by the researchers using a Brunton Atmospheric Data Center, or ADC Pro Model which measures air temperature, relative humidity, barometric pressure, and wind speed. These were noted periodically. Concrete temperature was only monitored during QA/QC sample tests and research sampling and testing.

Concrete samples for the material testing portion were taken by the research team or by AHTD personnel from the concrete pump discharge. This was done on the deck, or at the same level and as close to the active placement as possible, to provide an accurate measure of the concrete fresh properties as placed while minimizing disruption to the contractor's operation. The fresh concrete tests performed were slump (AASHTO T 119), unit weight (AASHTO T 121), air content (AASHTO T 152), and concrete temperature (AASHTO T 309). For the first three decks, the research team performed all the fresh concrete tests at three different locations (beginning, middle, and end regions) on the bridge deck and cast 4" x 8" cylinders for compressive strength tests at those same locations. At the middle sampling location, other hardened property specimens were also cast. The last two decks were smaller, and therefore the research team performed the fresh concrete tests and cast compressive strength cylinders at only two locations on the bridge decks.

The specimens cast at the first bridge deck were transported the morning after the deck placement, therefore complying with AASHTO T 23. However, for the four other bridge decks, the majority of the samples were transported before the eight hour minimum time limit. These samples were transported in the back of a full-size truck in containers built to restrict movement that were placed on approximately three inches of soft foam to reduce vibration.

5.2.2 Crack Mapping

Each bridge was revisited sometime after casting to assess the cracking. A representative area of deck was manually surveyed and the cracks mapped in the same manner as described in Chapter 4. One deck was mapped prior to opening to traffic. The others required AHTD to provide traffic control. Details and results for each bridge are described below.

5.2.3 The Bridges

The field study was coordinated with AHTD Research section and Resident Engineers. Active AHTD bridge construction sites where concrete deck placement was imminent were identified. During the summer construction season of 2005, five sites were chosen, shown in Figure 5.2.1 and listed in Table 5.2.1 below. The sites were selected based on sampling as broad a spectrum of bridge construction as possible in terms of geography, bridge type and size, and different contractors. All the bridges chosen were continuous steel girders designed to meet the requirements of AASHTO bridge design specifications. The design specification for each is shown in Table 5.2.1. All were designed by the Bridge Division of AHTD, except bridge 2, which was designed by consultant. All bridges used composite girders, made composite with the slab by welded shear connectors on the top flange, and galvanized steel stay-in-place formwork between beam lines.

AHTD allows the contractors the option of continuous pours for bridge decks. Rather than casting the positive moment regions of the bridge deck first followed by the negative moment regions, most contractors are choosing continuous casting or pours to speed up the construction process, as there are mandated delays between casting sections. If the contractors choose this option, the concrete must remain plastic during the entire length of the pour. This typically requires the use of a set-retarding agent in the mixture. In this research program, all of the contractors had elected to make these pours continuous.

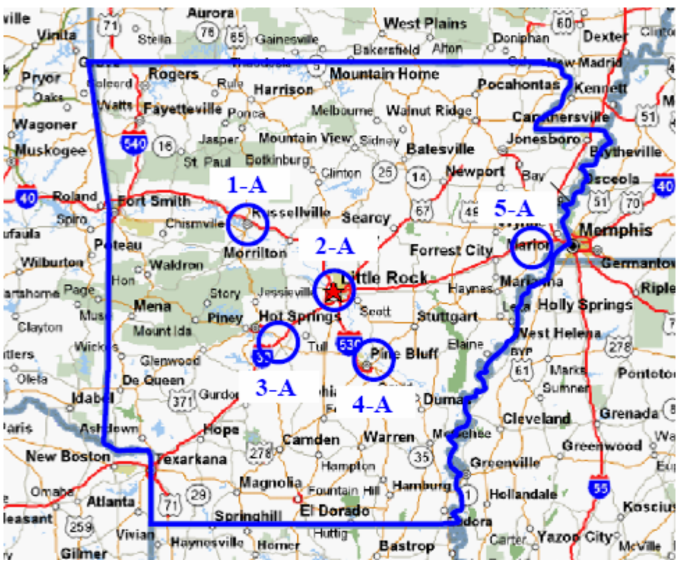

Figure 5.2.1 Map of Bridge Deck Locations

Table 5.2.1 Field Study Bridge Decks

Bridge ID	Job No.	Bridge No.	Bridge Description	Design Specification
1 - A	R80072	07007	Mill Creek Rd. over I-40	L.F.D., 17th Ed., 2002
2 - A	B60117	B6909	I-40 over Hwy 365, 176, RR	L.F.D., 16th Ed., 1996
3 - A	060938	04875	Rd. over Ouachita River	L.F.D., 17th Ed., 2002
4 - A	020384	07024	S.H. 15 over Main Ditch	L.R.F.D., 3rd Ed., 2004
5 - A	110388	07016	U.S. 70 over Bevins Slough	L.F.D., 17th Ed., 2002

5.2.4 Concrete Mixtures

All the bridge decks were constructed using AHTD Class S (AE) concrete. The mixture proportions are listed for all the decks in Table 5.2.2. As previously stated, AHTD requires a maximum w/cm of 0.44, a total cementitious material content of 611 lb/yd^3, and an air content of 6±2% for Class S (AE) concrete. AHTD allows the use of fly ash and slag cement in bridge decks. The maximum fly ash replacement rate is 20% by weight, and fly ash can either be Class C or F, with no mixing of the two. Fly ash was the only supplementary cementing material (SCM) used in the bridge decks. Admixtures used must be on the AHTD Qualified Products List (QPL), be used according to the manufacturer's specification and produce satisfactory results in test batches produced for mix design approval. AHTD does not specify a coarse aggregate content, but a coarse aggregate meeting either the AHTD Standard Gradation or the AASHTO M43 #57 Gradation must be used.

Table 5.2.2 Concrete Mixture Proportions

Materials	Bridge Decks				
	1	2	3	4	5
Cement (lb/yd^3)	519	519	489	611	611
Fly Ash (lb/yd^3)	92	92	122	0	0
Fly Ash (%)	15	15	20	0	0
Coarse Agg. (lb/yd^3)	1670	1670	1940	1722	1749
Coarse Aggregate Type	Limestone	Limestone	River Gr.	Limestone	River Gr.
Fine Aggregate (lb/yd^3)	1293	1293	978	1273	1112
Water (lb/yd^3)	269	269	251	269	298
w/cm	0.44	0.44	0.41	0.43	0.44
AEA Dosage (fl. oz/cwt)	0.75	0.70	1.00	0.50	0.65
Set Retarder Dosage (fl. oz/cwt)/Product*	8.0 Daratard 17	6.87 Recover	1.53 – 2.0 MB300R	2.81 Daratard 17	3.0 Daratard 17

* Daratard 17 and Recover are produced by Grace Construction Products;
MB300R (Pozzolith 300R) is produced by BASF Admixtures.

All five decks were cast from concrete produced at off-site ready-mix plants and delivered in rotating-drum mixer trucks. All five pours were placed using a concrete pump truck to deliver the concrete from the ready-mix truck to the deck surface. The concrete was vibrated using hand-held electric vibrators. The concrete was screeded to grade and initially finished by a series of augers, rollers, and float-pans moving transversely on a frame that advances longitudinally along the bridge deck. This finishing machine is supported by rails mounted on the exterior girder or on the overhang form supports. The entire construction sequence for each bridge is detailed below.

5.3 Bridge Deck 1

The first bridge deck visited is an interstate overpass. The bridge is 272 feet long and 43 feet wide and is a 2-span plate-girder bridge with spans of 149 feet and 123 feet. The bridge has a 30 degree skew. The intermediate support, located in the median of the interstate, and the abutments have elastomeric bearings under each girder to allow for rotation. The abutment bearings have slotted holes for the anchor bolts and the deck has poured-silicone joints at both ends to allow for thermal movements. The bearings at the intermediate pier do not allow for significant longitudinal movement. These are termed "fixed" bearings in AHTD bridge plans. The bridge deck has a 5.15% grade longitudinally and transitions from a normal 2% crown to over 4% constant superelevation. The deck is eight inches thick and supported on five welded-plate girders with 48 inch deep webs, spaced nine feet on center.

The bridge deck was cast June 15, 2005, and concrete placement began at 5:45 AM. Skies were clear during the placement. The air temperature ranged from 68 °F at

5:45 AM to 95 °F at 2:10 PM. The relative humidity ranged from 47-52 %. Only light wind, 3 – 5 mph gusts, was recorded, mostly after concrete was placed.

The concrete was pumped onto the deck and deposited in front of the finishing machine supported on rails attached to the overhang form supports. One construction worker with a commercial pressure washer fogged the concrete in the area of placement. The concrete was then vibrated, screeded, floated with a pan attached to the finishing machine, and then manually tined with a rake. Tining occurred approximately 30 minutes to 1 hour after finishing. The finish was generally good and uniform, except near mid-span of the second span. Pump malfunctions caused delays to the placement which resulted in the concrete drying more in this area prior to tining. This resulted in "balling", or bits of concrete collecting on the surface as it is tined. Curing compound was applied from a work bridge using an electric sprayer drawing from a drum of compound. The curing compound was sprayed approximately 30 minutes to 1 hour after finishing. The concrete was in place at 12:20 P.M. The total quantity of concrete placed was 331 yards, at average rate of 50.3 yards/hour.

No final curing materials were placed at 2:10 pm and concrete was still soft enough to be marred with a finger tip in some locations. At 7:30 A.M., June 16, 2005, the deck was covered with cotton mats and burlap, and workers were laying out the last of the plastic sheeting. The temperature was approximately 78 °F and the sky was cloudy. Slab joints were sawn in designated locations near the intermediate bents later that day. The contractor maintained the wet cure for seven days by wetting the burlap down using a water truck once or twice a day, and then recovering with the polyethylene sheeting.

On the seventh day, the plastic sheeting was removed and the burlap was allowed to dry and then removed.

Noteworthy during the placement was the vertical movement of the bridge due to interstate traffic passing underneath. Standing on the rebar mat in front of the screed, the movement was noticeable and remarkable to the researchers and other bridge engineers present for the placement. No comments from the RE representatives or contractors personnel indicated that it was out of the ordinary. In subsequent visits to the deck, when standing on the hardened concrete, the movement was reduced, but still noticeable.

5.4 Bridge Deck 2

The second bridge deck visited was a portion of an interstate bridge over two local roads and two railroad tracks. The portion cast was two spans of a 4-span continuous unit that is part of a two unit bridge. The bridge was built using staged construction and the portion cast on this date was the second stage. The placement width was 32 feet 6 inches from the outside rail to the edge of the stage 1 slab. The spans placed were 150 feet and 180 feet as measured along the centerline of the bridge, but actual lengths were shorter, as this stage was to the inside of a two-degree curve. Support lines at the piers for these spans were radial to the centerline curve and all girders were supported on elastomeric bearings. The bearings at the abutment where the placement began and the first pier allow guided longitudinal expansion movement, while the pier where the placement stopped had “fixed” bearings, which provide a pinned connection to the pier. The deck thickness was eight inches and was supported on 64 inch deep, welded plate-girders, seven foot six inches on center. The deck had a 6.2 % cross-slope toward the outside rail. All deck reinforcing was continued transversely from the previous stage and longitudinally over the pier to the subsequent deck sections. The joints created by the stage construction and pouring sequence would later be sealed with the same sealer as transverse slab joints sawn to provide for crack control.

The pour was made overnight from 9:00 PM, July 20 to 3:05 AM July 21, 2005. The total quantity of concrete used was 330 cubic yards and the deck was a continuous pour. The placement rate averaged 55 cubic yards per hour, although this does include some 20 minute delays waiting for concrete trucks to arrive. The weather conditions were hot and humid, with clouds early but clearing as the pour continued. Temperatures ranged from 89 at the start to 83 at the completion. The relative humidity climbed from 61% to 78%. No measurable wind was recorded. Interstate traffic was constricted to one lane next to the median rail on the slab from the previous stage. The resulting traffic vibrations could be felt in the area of the placement. Although a law enforcement vehicle was stationed near the construction activities most of the time, traffic speeds routinely surpassed the reduced speed limit when the officer was not there.

The concrete was pumped up to the deck. One construction worker fogged the concrete at the surface near the finishing machine, prior to floating, using a water nozzle connected to a compressed air line. The concrete was screeded and floated with a pan attached to the finishing machine, which was supported on the overhang form supports on the outside and on the first girder away from the hardened concrete from the previous stage on the inside. This left a narrow strip between the area the large machine could finish and the previous stage concrete. This portion was screeded using a small air operated screed supported on edge of the hardened concrete and the rail supporting the larger machine. The concrete was then bull floated with a 10-foot, rounded float, called the "highway screed", which caused a fair amount of paste to be drawn to the low-side gutterline. The concrete was then manually tined with a rake approximately 20 – 30 minutes after it was floated. The concrete was sprayed with a curing compound using an electric pump sprayer, beginning three hours after placement began. The curing compound application was stopped after the spray began marring the concrete surface finish. The curing compound was resumed and continued to completion after another three hour delay. Only one application was made.

Later in the morning of July 21st, the deck was covered with poly-burlap for final cure, seven hours after placement was completed. Deck joints were sawn on July 21st prior to fully covering the deck. The wet cure was maintained for seven days using soaker hoses attached to municipal water supply placed over the burlap. Temperatures during this time were in the mid-nineties and the job site received some rain.

5.5 Bridge Deck 3

The third deck was a 3-span unit of a 2–unit bridge that spanned a river carrying a city street. The 367-foot unit has spans of 113 feet, 141 feet, and 113 feet. The width is 43 feet total for a 40-foot clear roadway. The bridge has a less-than-1% longitudinal grade, a 25 degree skew, and a peaked crown, with a normal 2% slope to the gutterlines. Elastomeric bearings support each beam. One intermediate support had "fixed" bearings while the bearings at the other piers allowed for longitudinal thermal movements. Joints at each abutment and at the junction of the two units also allow for thermal expansions and contraction. The deck has a structural thickness of eight inches and is supported on five 60 inch-deep welded plate girders spaced nine feet on center.

The placement consisted of 400 cubic yards of concrete and was a continuous pour. The deck was cast August 24, 2005 beginning at 3:15 AM and finishing at 12:20 PM. The weather was hot and humid, with air temperatures of 76 – 95 °F and relative humidity readings of 73 to 59%. Skies were partly cloudy and little measurable wind was recorded.

The concrete was screeded with the finishing machine, supported on the overhang form support brackets, and floated with a pan attached to the finishing machine. Like the previous decks, one construction worker fogged the concrete near the finishing machine using a pressure washer. The deck surface was then bull floated with a “highway float”, and then tined with a finned float, 30 minutes to one hour after being finished. The concrete was then sprayed with curing compound, although this did not commence until six hours after the first concrete was placed and was accomplished with a manual pump sprayer from one side of the deck.

Final cure consisted of poly-burlap mats being placed dry and then wetted, 3.50 hours after the placement was complete. The wet cure was maintained for seven days using water from tank pumped on the deck. Weather conditions during this period were similar to those on the day of placement.

5.6 Bridge Deck 4

The fourth bridge deck was a state highway bridge that spanned a drainage ditch. The three-span bridge is a 124-foot, continuous, rolled wide-flanged beam bridge with spans of 38, 48, and 38 feet. The bridge deck is 33 feet wide and has a 28-foot clear roadway. The longitudinal grade is level and the deck slopes down 2 % from the centerline to each gutter. The bridge has a 15 degree skew. The intermediate pier bearings are "fixed" bearings. The abutment bearings are slotted and the deck has joints at each end to allow for thermal movement. The deck has a structural thickness of 8 ¼ inches and is composite to the four wide-flange beams, spaced nine feet four inches on center.

The deck was placed September 7, 2005 from 6:00 AM to 10:25 AM. The placement consisted of a 117 cubic yard continuous pour. Weather conditions were clear and air temperature climbed from 67 °F at the beginning to 94 °F at completion. The relative humidity ranged from 70 to 39 % during the placement.

The concrete was pumped, screeded with the finishing machine, supported on the overhang form support brackets, and then floated with a pan and dragged with burlap that were both attached to the finishing machine. It was then tined with a rake, but with inconsistent results. At times the tining was delayed an hour or two because the concrete was too wet. Once resumed, however, "balling", where small balls of concrete paste collect on surface along the tines, occurred due to the concrete being too dry at the surface.

The surface was then sprayed with curing compound, from a work bridge using an electric sprayer. Curing compound was applied three hours after finishing. Final cure was applied using the poly-burlap mats, rolled out dry five to six hours after the placement was complete and after the slab joints had been sawn in the deck surface. The mats were wetted with water pumped from the drainage ditch below. The cure was maintained using the same water supply for seven days, during which the weather was similar to that the day of the placement.

5.7 Bridge Deck 5

The final bridge deck is a US highway spanning a small creek. The bridge was a 175-foot, three span, continuous, rolled-beam unit with spans of 55, 65, and 55 feet. The bridge is straight and square and has a level grade and normal crown. This bridge has integral supports, made so by casting a concrete diaphragm around the beams at each support. These were cast prior to casting the deck slab, but are made continuous with the slab by reinforcing steel stirrups that connect them. The slab is 7 ½ inches thick and supported by five wide-flange beams spaced seven feet six inches on center. The bridge deck is also made continuous with the approach slabs by reinforcing bars that are connected through the abutment. This is typically done for bridges in the seismic zone in Arkansas, and leaves no expansion joint, only a small gap sealed with polymer joint material, at the bridge ends

The bridge deck was placed on September 23, 2005. The placement began at 7:05 AM and all concrete was in place at 10:40 AM. The deck was a 171 cubic yard continuous placement, at an average rate of 47.7 cubic yards per hour. Weather conditions were clear with air temperatures ranging from 71 – 98 °F and relative humidity falling from 67 to 42 %. Light wind was measured later in the day, with gusts of 5 – 10 mph.

The concrete was pumped; screeded with the finishing machine, supported on the exterior girders; and floated with two pans attached to the finishing machine. The deck was bull floated with a 10-ft. rounded float, and then manually dragged with burlap. It was then tined with a rake, 25 – 35 minutes after finishing. The deck was then sprayed with curing compound using an electric pump sprayer, approximately 1 ½ to 2 ½ hours after finishing.

Saw cutting of crack control joints began 1 ½ hours after the last of the curing compound was sprayed. Final wet curing was placed 4 ½ hours after the concrete was placed. It was comprised of dry poly-burlap blankets being rolled out onto the surface and then wetted using water pumped from the creek. This system was used to wet the burlap daily for the next 10 days. At which time, the blankets were removed. During this period, the air temperature stayed in the upper 80s and some rain fell.

5.8 Concrete Data

5.8.1 Fresh Concrete Properties

As stated in the Testing Program, the fresh concrete properties were measured in two or three random locations (determined by AHTD) for each bridge deck. If the bridge deck was large enough, the sampling locations were typically at the beginning, middle, and ends of the bridge deck. The results from all the fresh concrete tests and the AHTD specifications for each property are shown in Table 5.8.1.

From Table 5.8.1, one can see that the four of the five bridge decks had slumps that exceeded AHTD specifications in at least one location. Bridge Deck 1 was the only deck where all slumps fell within the 1 to 4 inch specification. For the air content, three of the five bridge decks had measured air contents that did not meet AHTD specifications. Only two bridge decks had fresh concrete temperatures that were greater than that allowed by AHTD.

The final fresh concrete properties shown in Table 5.8.1 are the calculated and measured unit weights. The calculated unit weights are based on the concrete mixture proportion used by the concrete supplier and assuming a fresh concrete air content of 6%. The differences between calculated and measured unit weights ranged from a low of 1 lb/ft^3 to a high of 9 lb/ft^3. These differences between calculated and measured unit weights could be attributed to the addition of extra mixing water or to higher or lower than expected air contents.

Table 5.8.1 Fresh Concrete Properties

Bridge Deck	Slump (in)	Air Content (%)	Calculated Unit Wt. (lb/ft^3)	Measured Unit Wt. (lb/ft^3)	Concrete Temperature (°F)
1	3.50	5.8	140	143	83
	3.25	6.3		141	89
	2.75	4.9		[2]	90
2	4.50	3.8	140	148	95
	7.25	3.5		146	92
	2.50	2.2		149	95
3	6.25	3.2	140	144	92
	3.50	4.6		[2]	83
	5.00	5.0		145	95
4	8.25	9.2	141	136	73
	6.00	8.7		138	81
5	6.00	5.7	139	141	80
	3.50	4.8		144	86
AHTD Specifications	1-4	4-8	None	None	40°-90°

5.8.2 Compressive Strength

The results from the compressive strength tests are shown in Table 5.8.2. All of the concrete mixtures used had design strengths of 4000 psi at 28 days. Bridge deck 1 had the lowest one day strengths. The first and last sampling location had a one day compressive strength of approximately 300 psi while the middle sampling location had a one day compressive strength of 60 psi. At two days of age, cylinders that were sampled from the first and middle locations of the bridge deck were tested. These tests showed that the first location had gained over 2000 psi in 24 hours, but the middle section was still much lower (a compressive strength of 130 psi). By 28 days and 56 days of age, the middle section had reached similar strengths as the first and last sections of the bridge.

The only other bridge deck to have large variations in compressive strength was Bridge Deck 4. This deck was a smaller pour and due to time constraints only a limited number of cylinders were sampled from the last portion of the deck. As seen in Table 5.8.2, the compressive strengths at one and 28 days of age for the last section of the deck were much higher than the first two sections (over 2000 psi at one day and over 4000 psi at 28 days). For this particular bridge deck, the concrete supplier was having problems with the air content. For the first two sections the air contents were at or near 9%, and efforts were being made to lower the air contents. If the air content was indeed lower for the last section, this should result in the higher compressive strengths.

Table 5.8.2 Compressive Strength Results[1, 2]

Bridge Deck	1 Day	2 Day	7 Day	28 Day	56 Day
1	340	2590	-	6680	6850
	60	130	-	6370	7000
	320	-	5750	7540	7950
2	-	3420	5100	5070	5840
	-	2770	4530	5050	5260
	-	3570	5780	5830	6930
3	1480	-	3670	4400	4640
	2140	-	3590	4190	4530
	1940	-	3830	4710	4950
4	2350	-	3780	4940	4980
	2860	-	4430	5520	5560
	4730	-	-	9120	-
5	2960	-	4010	4660	4370
	3630	-	4410	5330	5710

[1] For each deck, each row represents one sampling location.
[2] These strengths represent the average of three tests.

5.8.3 Rapid Chloride Ion Penetrability

The permeability of the hardened concrete sampled from the bridge decks was measured by the Rapid Chloride Ion Permeability (RCIP) tests (AASHTO T 277). The results of the 28 and 90 day RCIP tests for all bridge decks are listed in Table 5.8.3 along with the permeability classes based on AASHTO T 277. The results are the averages of four test specimens.

Table 5.8.3 RCIP and Freeze/Thaw Results for Field Study

Bridge Deck	28-Day RCIP (coulombs)	Permeability Class	90-Day RCIP (coulombs)	Permeability Class	Freeze/Thaw Durability (DF)
1	2807	Moderate	2551	Moderate	95
2	4019	High	2898	Moderate	101
3	5300	High	4072	High	86
4	2552	Moderate	3047	Moderate	105
5	2424	Moderate	2429	Moderate	47

Decks 1, 4, and 5 had RCIPs in the moderate range (2000-4000 coulombs) at 28 days and Decks 2 and 3 had RCIPs in the high range (>4000 coulombs) at 28 days. At 90 days, Deck 3 was the only deck in the high permeability class and all the other decks had moderate permeability. The 90-day permeability for the first three decks decreased from the 28-day permeability. Decks 4 and 5 had 90-day RCIP values greater than the 28-day values. However, the 90-day RCIPs were still in the moderate permeability class. Since the specimens were air cured, the RCIP might have increased due to a lack of water to continue hydration.

The RCIP of Decks 1 and 2 should theoretically be similar, due to the same mixture proportions. However, minor differences in RCIP values could be due to differences in materials. Since the concrete from the two decks had different amounts of field water added and were batched at different plants, there could be more significant differences in RCIP. The concrete sampled for the RCIP test for Deck 1 had a slump of 3.25 inches and the concrete sampled for the RCIP test for Deck 2 had a slump of 7.25 inches. The RCIP values at 28 days are very different for the two decks. For Deck 1, the 28 day RCIP value is 2807 coulombs and 4019 coulombs for Deck 2. This is a difference of 1212 coulombs and puts the decks in two different permeability classes (moderate for Deck 1 and high for Deck 2). The higher slump for Deck 2-A most likely relates to a higher w/cm and therefore might be the reason for the higher RCIP. However, at 90 days, the RCIP for both decks are nearly similar, 2551 coulombs for Deck 1 and 2898 coulombs for Deck 2.

Theoretically, Deck 3 should have had the lowest RCIP due to the lower w/cm and higher replacement rate of fly ash as compared to the other decks with higher w/cm and lower or no fly ash content. However, Deck 3 had the highest RCIP at 28 and 90 days. Deck 3 did have the second highest average amount of field water added, which means that the w/cm was most likely raised for most of the concrete placed for the deck. Decks 1, 2, and 4 had nearly the same mixture proportions, except that there was no fly ash replacement for Deck 4. Since the decks had different water contents due to many factors, the RCIP of Deck 4 cannot be compared to Decks 1 and 2 to determine if the addition of fly ash improved RCIP for the mixture. However at 90 days, Deck 4 was in the high permeability class and Decks 1 and 2 were in the moderate permeability class. Deck 5A had the lowest permeability at 28 and 90 days.

5.8.4 Freeze/Thaw Durability

The freeze/thaw durability of the specimens from the bridge decks was determined by monitoring the resonance frequency of specimens sampled from the decks. The first and last frequencies (after approximately 300 cycles) recorded for each specimen were used in determining the durability factor as per ASTM C 666-A. Table 5.8.3 lists the average durability factors of the concrete sampled from the bridge decks.

The first four decks had durability factors over 60. Deck 5 was the only deck not to have a durability factor over 60. Most researchers agree that a durability factor of 60 or above is adequate for freeze/thaw durability. Since all the decks should have had adequate air entrainment, the concrete should have a durability factor greater than 60. Decks 2 and 4 had durability factors greater than 100. This is most likely due to the specimens still gaining strength after being subjected to freeze/thaw cycles.

Even though Deck 2 had an air content lower than the AHTD specified minimum of 4%, the durability factor was well above 60. The lower w/cm and increased fly ash for Deck 3-A appears to not have increased durability as compared to the other decks with higher w/cm and lower or no fly ash content. Deck 4 had the highest air content and the highest durability factor, which is expected. The aggregates in Deck 5 deteriorated throughout the freeze/thaw cycles and created cracks in the paste of the specimens. This was the cause of the lower durability factor for Deck 5.

5.8.5 Unrestrained Shrinkage

Unrestrained shrinkage of the decks was measured at 1, 4, 7, 14, 28, 56, 112, and 224 days of age. The unrestrained shrinkage test results for decks 2, 3, 4, and 5 are listed in Table 5.8.4. These values are the average of four specimens.

Table 5.8.4 Unrestrained Shrinkage Results for Field Study

Age (days)	Bridge Deck			
	2-A (microstrains)	3-A (microstrains)	4-A (microstrains)	5-A (microstrains)
4	41	68	68	44
7	87	104	89	84
14	211	179	168	103
28	257	218	234	182
56	348	297	411	273
112	451	447	492	339
224	498	434	444	303

Shrinkage values for Deck 1 were measured with a faulty dial gauge comparator and the results were erroneous. A different digital gauge comparator was used for the other four decks. Up to seven days, the rates of shrinkage (only 24 microstrains difference between the low and high) were nearly the same for all decks. After seven days, the rate of shrinkage for Deck 5 decreased much more than the other decks. By 56 days, the majority of shrinkage occurred for all four decks. Between 28 and 56 days, nearly half the shrinkage of Deck 4 occurred. Decks 3, 4, and 5 both shortened between 112 and 224 days by 13 microstrains, 48 microstrains, and 36 microstrains, respectively. The 112 day measurements were taken in December for Decks 3and 4 and January for Deck 5. Both months typically have low relative humidity in the state of Arkansas. The 224 day measurements were taken in April for Deck 3 and May for Decks 4 and 5. April and May typically have a higher relative humidity than December and January in Arkansas. Therefore, the increase in length is most likely due to an increase in relative humidity in the environmental chamber. Deck 2 had the highest slump and the highest overall shrinkage. The high slump is most likely due to increased water content. Paste (cement and water) is the largest contributor to drying shrinkage. Therefore, the high amount of shrinkage for Deck 2 is most likely due to increased water content. Deck 4 had the highest maximum amount of field water added and the second highest amount of overall shrinkage.

5.9 Crack Mapping

As previously described, each bridge was revisited sometime after casting to assess the cracking. A representative area of deck was manually surveyed and the cracks mapped in the same manner as described in Chapter 4. One deck was mapped prior to opening to traffic. The others required AHTD to provide traffic control. The results of the surveys are summarized in Table 5.9.1. Details and results for each bridge are described below. The crack density reported is calculated as linear feet of crack divided by the survey area.

Table 5.9.1 Cracking Measurement Results

Bridge Deck	Survey Length (ft.)	Survey Width (ft.)	Survey Area (ft^2)	No. of Cracks	Lin. Ft. of Cracks (ft)	Crack Density (ft/ft^2)	Avg. Crack Width (in.)
1	50	12.0	600	80	235.8	0.393	0.013
2	330	31.4	10367	16	128.3	0.012	0.0024
3	100	12.0	1200	20	135.0	0.112	0.006
4	100	12.0	1200	6	7.42	0.006	0.0089
5	65	12.0	780	10	39.5	0.051	0.0046

5.9.1 Bridge Deck 1

Bridge deck 1 was revisited June 24, 2005, after the curing blankets were removed. Cracking was already evident, especially in the area near the intermediate pier and along the slab edges. The cracking was not mapped completely at this time due to contractor’s equipment and materials obscuring most of the deck. However, some crack widths were measured and some photos were taken. The crack widths ranged from 0.012 inches to 0.047 inches. The longer cracks were mostly longitudinal, with many originating near the transverse slab joint at the intermediate pier. Some cracks appeared to have occurred prior to the sawing of the joint as evidenced by their continuation across the joint. Cracking in the gutter areas was more random in direction; however, some were approximately parallel to the skew direction. They were heaviest in the smooth finished section between the tined area and the protruding rail reinforcing steel. This area was hand troweled during the placement, received curing compound at similar coverage to the tined areas, and was covered with the same curing materials. Some of these cracks continued over edge and down the thickness of the slab.

The cracks were mapped on April 5, 2006 after the bridge was open to traffic and after the contractor had sealed larger cracks at some time prior to opening. The researchers attempted to map all cracks in a 12 by 100 foot section of south bound lane, but after measuring 40 feet of the 12 foot wide section, cracking became too small and random to effectively map. A 10 foot by 12 foot sub-area was measured as a representative sample. From visual estimation, the density was approximately the same as the representative sample for the remainder of original 100 foot section, although it lessened some in last 15 feet. The cracks ranged from four inches to 48 feet in length and from less than 0.005 to 0.024 inches in width. The cracks were a mix of transverse and longitudinal cracks with diagonal connecting cracks. Long lines of cracking in the wheel path of the lanes were observed. As noted above, cracks were concentrated over the center support (near the middle sampling location) of the deck. This could possibly be due to a combination of vibrations from traffic passing under the bridge, which were noticeable, and low compressive strengths, at least up to seven days, at this section.

5.9.2 Bridge Deck 2

Bridge deck 2 was revisited on September 1, 2005, after the curing blankets had been removed. The visible cracks were measured for the whole pour. The cracks ranged from three inches to 17 feet in length and 0.002 to 0.016 inches in width. The cracks were mostly transverse and fairly heavy in the positive moment section of the second span. These cracks were located mainly toward the middle of the section placed. These cracks did not appear to be plastic shrinkage cracks as the edges were cleanly fractured, not ragged as cracks occuring in fresh conrete. Although, the locations of the initial point of cracking for the individual cracks was not determined, thus, the cause of the initial crack cannot be determined for certain. These cracks were similar in orientation and had similar starting and stopping points, suggesting some relation to the beam lines underneath.

There were other cracks that did look like the random, ragged cracks associated with plastic shrinkage. The plastic shrinkage cracks were located near the low gutter (the downhill side of the deck). Large amounts of paste were brought down to this side during construction using a highway screed. High amounts of paste might have contributed to increased shrinkage in that area.

5.9.3 Bridge Deck 3

Bridge Deck 3 was revisited on January 27, 2006, prior to being opened to traffic. The bridge opening was delayed due to unresolved deck cracking issues in the second unit, placed a week after the placement monitored for this research. The research team measured the cracking in a 12 foot by 100 foot section of the west bound lane on the unit monitored during placement. The weather at the time of mapping was overcast with air temperatures in the low forties. The cracks ranged from three feet to 12 feet in length and were less than 0.007 inches wide, although the cold weather prior to mapping could have caused the cracks to be narrower. The cracks were almost exclusively transverse cracks that started and stopped at similar points in the cross section. These points correspond roughly to the beam lines supporting the bridge. The cracks tended to originate near the centerline of the bridge and terminate near the next girder, or start from that girder line and continue outward to the outside most girder. The cracks occurred approximately every six to eight feet longitudinally in the survey area. There is no clear indication as to the cause of the longitudinal spacing.

Another area outside of the survey area on bridge deck 3 was noticed to have a bubbled or pitted surface. It was most notable in a two to three feet wide by four to five feet long patch near the crack control joints adjacent to the pier in the first span, centered in the traffic lane. Fresh property tests show that the air content was not out of the specified range. The surface tining was also found to be shallow in the first 25 feet from the abutment-end of the unit-1 deck. The deck of unit 2, placed a week after unit 1, experienced delays due to equipment failures during the placement and had cracking problems, as well. A visual inspection of this deck showed the cracking to be generally short, longitudinal cracks with fewer transverse, lane width cracks than unit 1. The tining and general finish were also found to be very shallow and inconsistent, which suggests difficulty in getting the concrete finished and tined prior to the set of the concrete. AHTD requested the contractor seal the deck, although the type of material to be used and extents of any sealing were be debated among the parties.

5.9.4 Bridge Deck 4

Bridge deck 4 was revisited on February 9, 2006, after the bridge had been open for traffic for a couple of months. The research team measured cracking in a 12 foot by 100 foot section of the deck. There was very little cracking in the deck. The cracks ranged from six inches to four feet in length and 0.002 to 0.010 inches in width. Some cracks exhibited the ragged, torn appearance of plastic shrinkage in one location, but looked as though they were beginning to extend from this point. In some areas of the deck, passes of the tinning rake were overlapped by one to two inches during the placement. In these areas, some small tears, or pulls in the surface of the fresh deck were observed in the hardened deck. There were also very small cracks, 0.5 inch or less in length and 0.005 inch or less in width, perpendicular to the tining in the top surface of the ridges of the tines, occurring of the whole deck. These hairline cracks did not seem to be as deep as the trough of the tine, but some did extend to the adjacent ridge. These may represent a localized shrinkage in the paste at the surface, but not a severe flaw in the concrete deck.

Areas near the rail of bridge deck 4 also exhibited cracking. The cracks appeared to initiate in the rail itself, at the bottom of sawn joints in the rail. This occurred at every rail joint, and in some locations, these cracks in the face of the rail continued in the smooth-finished portion of the slab. A few cracks in the slab surface also were observed in the smooth areas near the drain openings in the rail. These cracks extended only to the edge of the tined surface and were less than 0.005 inches in width. This was observed outside the survey areas. The rails of this deck were cast some time after the deck, a week minimum, and were likely slip-formed.

5.9.5 Bridge Deck 5

Bridge deck 5 was mapped on February 10, 2006. The bridge was open to traffic and the weather was cold and cloudy. The air temperatures were in the low 40s and precipitation began to fall as the mapping began. The cracks were measured only over a 12 foot by 65 foot section due to declining weather conditions. The cracks mapped were 2 feet to 5 feet in length and were 0.002 to 0.01 inches wide. The longer cracks were mostly transverse and similar to those on previous decks, in that they started and stopped at similar points in relation to the beam lines. There were some other cracks that were two to five feet long and were angled 45 degrees to the intermediate bents. These were the wider of the cracks, and are curious in their proximity and orientation to the integral pier. Similar cracks were not readily observed in similar locations at bents outside the mapped area.

5.10 Investigated Deck Variables

The variables monitored during the five deck placements can be compared to the measured crack density. In the following sets of tables, various groups of recorded factor values are listed for each bridge, with the bridges listed in order with respect to crack density, from most cracking to least. Trends, or the lack thereof, are discussed for each group.

5.10.1 Bridge Design Factors

Prior to the placement, choices made by the engineers and contractors can affect the cracking potential of a bridge deck. Geometric proportions of the bridge affect deflections experienced by the deck. Some of these values are reported for the decks in Table 5.10.1. Depth to span ratio is calculated as the nominal depth of the girder and slab divided by the maximum span length in inches, and has been a long standing criteria for girder depth with respect to deflection (AASHTO, 2004). Values greater than 0.027 were deemed acceptable under the 2004 LRFD Bridge Design Specifications.

Table 5.10.1 Bridge Design Factors

Bridge Deck	Amount of Cracking ft/ft^2	Max Span Length (ft.)	Girder Type	Depth /Span Ratio	Girder Spacing (ft.)	Slab Depth (in.)
1	0.393	149	WPG	0.033	9.0	8
3	0.112	141	WPG	0.042	9.0	8
5	0.051	65	WBM	0.048	7.5	7.5
2	0.012	180	WPG	0.035	7.5	8
4	0.006	48	WBM	0.067	9.33	8.25

Girder Types: WBM = Wide Flanged Rolled-shapes, WPG = Welded-Plate Girder.

The depth to span ratio for bridge deck 1 is the closest to the minimum value and had the most cracking. Although this bridge was designed to meet the required values for dead and live load deflection, its narrow margin over this common guideline for section depth could explain the movement felt on the deck, both before and after the deck was placed, and subsequently the cracking near the intermediate support. The other bridges do not seem to follow this pattern, and no other correlations appear in these variables.

5.10.2 Concrete Materials

Contractors' choice of aggregates and admixture dosages affect the concrete's performance. These values are reported for the decks in Table 5.10.2. Admixture dosages were taken from mix designs or batch ticket information supplied by the contractors to AHTD. Set retarder dosage range and set-delay information was taken from manufacturer's websites.

Table 5.10.2 Concrete Material Factors

Bridge Deck	Amount of Cracking ft/ft^2	Aggregate Type	Set Retarder Dosage (fl oz/cwt)	Manf. Dosage Range (fl oz/cwt)	Expected Set Delay (hours)
1	0.393	Limestone	8.0	2-8	2-4*
3	0.112	Gravel	1.53-2.0	3-5	1-5
5	0.051	Gravel	3.0	2-8	1-2*
2	0.012	Limestone	6.87	2-5	No info
4	0.006	Limestone	2.81	2-8	1-2*

*Based on dosage rate and air temperature at time of use

While aggregate type may have an impact of the long term shrinkage and creep characteristics of the concrete, it does not have a direct correlation to cracking in these five decks. Bridge deck 1 had the largest dose of retarder and had the most cracking, and bridge deck 4 used the second lowest dosage of set retarder and had the least cracking. Increased set retarder doses may cause the concrete to remain in a plastic state longer, thus exposing it to more risk of plastic shrinkage cracking, or it may simply delay the curing process, opening the door for said cracking. However, direct correlation among these decks is difficult because the various products have different effects at different rates of addition. It is of note that none of the contractors reduced their retarder dose over the course of the placement, to reduce the delay for concrete near the end of the pour. When asked why, one contractors' representative indicated that the set-retarders used also act as air-entraining agents, and adjustments to the retarder dosage could upset the air content of the concrete. As indicated in Table 5.8.1, this was already a difficult thing to control.

In an attempt to determine of there were any relationships between the fresh concrete properties and crack density, the average slump, air content, measured unit weights, differences between measured and calculated unit weights, and concrete temperature were plotted versus the crack density. Each bridge deck was ranked by each concrete property and assigned a ranking. For example, Bridge Deck 1 had an average slump of 3.17 inches which was the lowest average slump of the five decks, and therefore it received a ranking of "1". Likewise, Bridge Deck 4 had the greatest average slump (7.125 inches) and received a ranking of "5". Shown in the Figure 5.10.2A are the rankings for each fresh concrete property and crack density. The graph shows that Bridge Deck 1, which had the highest crack density of 0.315 ft/ft^2, did not have the greatest value for any of the fresh concrete properties. Bridge Deck 1 had the second highest air content, third highest concrete temperature, fourth highest unit weight, and was ranked last in unit weight difference and slump. For the concrete properties measured and bridge decks samples, there was no correlation between fresh concrete properties and crack density.

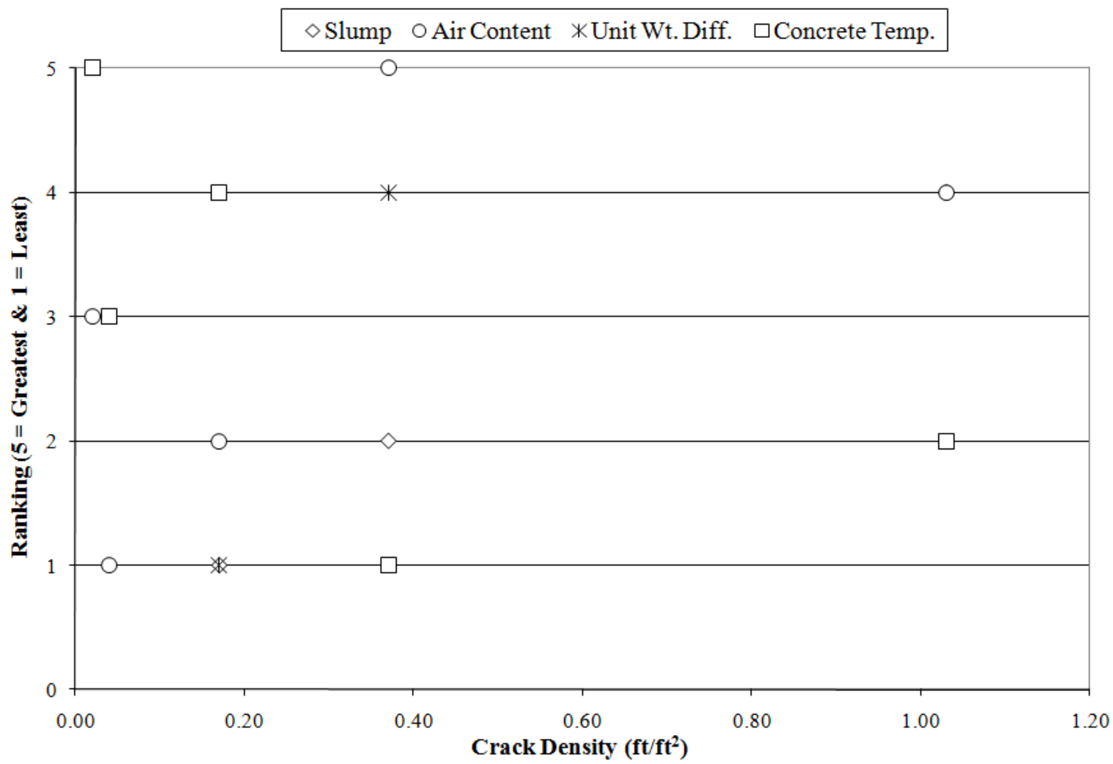

Figure 5.10.2A Fresh Concrete Properties vs. Crack Density

As with the fresh concrete properties, the hardened concrete properties were ranked and plotted versus the cracking density (Figure 5.10.2B) to determine if there were any relationships between the hardened properties and cracking for the decks in this study. Each hardened property was ranked from 1 to 5 and the rankings were plotted. Like the fresh concrete data, there were few if any correlations between the hardened properties and cracking. Bridge Deck 1 did have the greatest 7 and 28 day compressive strength and the most cracking, but Bridge Deck 3 which had the second highest crack density also had the lowest compressive strength at 7 and 28 days of age.

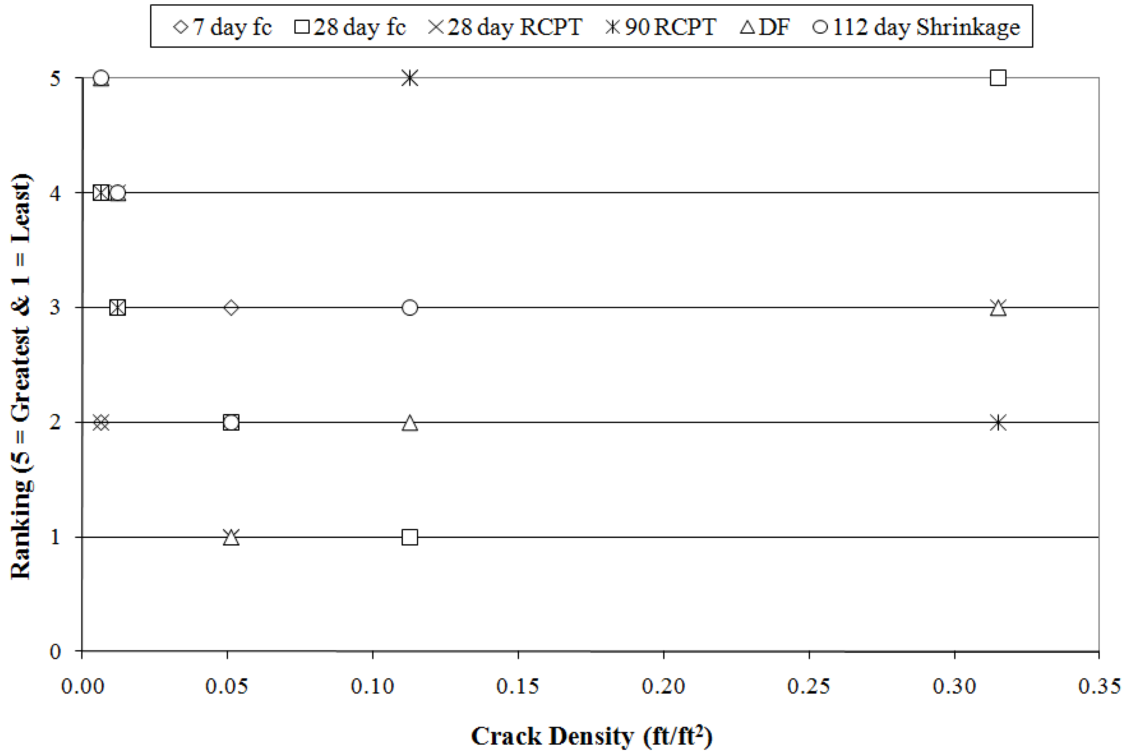

Figure 5.10.2B Hardened Concrete Properties vs. Crack Density

5.10.3 Deck Placement Variables

As the size of concrete deck placement increases, the effort required to maintain control over the numerous variables and factors also increases. If the contractors and inspectors are not prepared for this increase, the deck can be at a larger risk for problems. But, a smaller pour could also allow for slack in proper procedures and precautions that could expose the deck to increased risk. Some of the variables are explored for the five decks in Table 5.10.3 below.

Table 5.10.3 Deck Placement Variables

Bridge Deck	Amount of Cracking ft/ft^2	Concrete Volume (cu yds)	Placement Time (hours)	Placement Rate (cu yds/hr)
1	0.393	331	6.58	50.3
3	0.112	400	9.08	44.0
5	0.051	171	3.58	47.8
2	0.012	289	6.0	48.2
4	0.006	117	4.25	27.5

Bridge deck 4 was the smallest placement, but was placed at the slowest rate. However, the times and rates of placement shown here are for whole process, from the first delivery of concrete to the site to the finishing of the last concrete placed. This includes, in the case of bridge deck 4, delays due to delivered concrete that was too wet and rejected. In other decks, concrete plant equipment malfunctioned, causing delays as trucks were routed from other plants. The AHTD construction specification mandates a 20 cu. yd. per hour rate and delays of no more than 20 minutes between successive batches to prevent differential setting between batches. This can be difficult to maintain when problems arise on remote bridge sites.

When comparing the time of placement to the anticipated delay in set caused by the retarder dose, it can be shown that it is very difficult to maintain the entire volume of placed concrete in a plastic state until all the concrete has been placed and finished, as also mandated by AHTD construction specification. Assuming a 6 hour natural initial set time for this type of concrete (Kosmatka et al., 2003) and neglecting decreases in set times due to the high temperature of the concrete, decks 5, 4 and 1 could be finished prior to initial set. Deck 3 could have been very close to initial set by the time the placement was completed. However, in the field, all of the deck concrete was still easily marred at the completion of the placements. And deck 1, which had the most cracking, was still soft enough to be marred 15 hours later.

5.10.4 Environmental Conditions

The weather conditions at the time of the deck placement affect the cracking potential of the deck directly and indirectly. The weather readings taken during the placement should offer some insight as to the nature of these effects. Concrete temperature was recorded during the fresh tests, not after the concrete was in place, and therefore does not fully represent the temperature profile of the deck concrete due to heat of hydration. It does reflect the influence of initial concrete temperatures, which are difficult to regulate in the hot, summer construction season. Evaporation rates are calculated using the equation presented by Paul Uno (1998) and detailed in Ch. 6 and use the fresh concrete temperature recorded nearest to the time of the individual weather readings. The weather conditions are summarized in Table 5.10.4. Averages reported are the mean value of the readings recorded. Wind speeds are not included in this summary, because no significant sustained winds were recorded during any of the placements.

Table 5.10.4 Environmental Conditions

Bridge Deck	Amount of Cracking ft/ft^2	Air Temp. Range (°F)	Avg. Air Temp. (°F)	Ave. R. H (%)	Avg.Conc. Temp. (°F)	Evap. Rate Avg. /Peak (lb/ft^2/hr)
1	0.393	68-95	85.3	57	87.3	0.038/0.080
3	0.112	76-95	85.4	69	90.0	0.027/0.045
5	0.051	71-96	86.8	53	83.0	0.051/0.103
2	0.012	89-83	85.5	72	94.0	0.043/0.063
4	0.006	67-94	85.4	53	77.0	0.021/0.026

Bridge deck 4, with the lowest crack density, experienced the lowest mean and peak evaporation rate, the lowest average concrete temperature, and the lowest air temperature recorded during a placement. Bridge deck 1, with the highest crack density, however, falls to the middle of the group in all values. All evaporation rates calculated were well below the traditional reported 0.20 lb/ft^2/hr threshold for plastic shrinkage cracking, although this rate is the subject of debate in the literature. While they obviously contribute to it, favorable or unfavorable, ambient conditions do not solely determine the potential for cracking in a bridge deck.

5.10.5 Curing

The timing of curing operations can improve or undercut prior efforts to prevent plastic shrinkage cracking in a bridge deck placement. During the five bridge deck placements the times recorded were the times from concrete finishing to the application of curing compound and the time from completion of the placement to the application of the final cure. The results are summarized in Table 5.10.5.

Table 5.10.5 Curing Operations Timing

Bridge Deck	Amount of Cracking ft/ft^2	Time to Curing Compound Application (hours)	Time to Final Cure Application (hours)
1	0.393	0.5 -1	15.5
3	0.112	6	3.5
5	0.051	2.5	5.0
2	0.012	3	7.0
4	0.006	3	5.75

The time to curing compound applications for bridge deck 1 represents a consistent time frame from concrete finishing to the curing compound application for the entire placement. The times listed for the other four decks represent the time elapsed from the first finished concrete to the initial spraying of compound. The variations in pace of the placement operations and curing compound applications did not yield such a consistent interval for these decks. The early application of compound to bridge 1 did not prevent it from experiencing the most cracking. Bridge deck 3, with the second most cracking density, had the longest time to initial curing compound application. Bridge decks 2, 4, and 5 show no clear relationship between these variables exclusive of other factors. Manufacturers' directions instruct users to apply the compound when the free-water has left the surface of the fresh concrete. This time would be unique to each portion of placed concrete, so no standard interval exists for comparison.

The time to final curing would seem to indicate cracking potential, given the long delay for bridge deck 1. However, the remainder of the decks follow no pattern. The length of the final cure is not shown because all five decks met the minimum seven days required by AHTD specifications. Deck 5 was kept covered for 10 days, but this due to placement being on a Friday and blankets being removed on a Monday.

5.10.6 Cross – Variable Correlations

No solid single variable trends exist in the preceeding analysis. Analysis of the best and worst deck with respect to cracking may yield correlations between groups of variables.

Bridge deck 4 had the least cracking, the largest depth to span ratio, the second lowest set rearder dosage, the smallest volume of concrete, and second shortest placement duration, but the slowest rate of placement. Bridge deck 4 also had the lowest average concrete temperature and the lowest peak and average evaporation rate. It had the second longest time to curing compound application, the highest one day strength and second highest 28 day strength.

Bridge deck 1 had the most cracking, the second longest maximum span length, the smallest depth to span ratio, the largest set-retarder rate, and the longest anticipated set delay due to retarder. Bridge deck 1 was the second largest placement in terms of concrete volume and time duration, but the had the fastest placement rate. Deck 1 had the second highest peak evaporation rate, but the third average rate. Deck 1 had the shortest time to curing compound application but the longest to final cure. Its average one day strength was the lowest, but its 28 day average was the highest.

This suggests that the large volume of concrete required for bridge deck 1 took a long time to place, and therefore the set was delayed to the maximum degree possible using the chosen materials. During this period, the deck was exposed to peak evaporation rates and no final curing, despite early application of curing compound. This presented a risk for plastic and drying shrinkage cracking to begin. A flexible section and low initial concrete strengths did not resist the flexing from construction loads and uplift due to traffic passing underneath. This may have caused tensile stresses near the support in excess of the concrete's capacity, which could explain the increased cracking in this area.

Conversly, bridge deck 4, was placed rather quickly, even at the slowest rate of the decks. The weather conditions were not as harsh, resulting in a lower evaporation rate. This negated the effects of the later application of curing compound, and less delay to the concrete set meant that final curing could be applied in a moderate amount of time. After construction activity was complete, there was no substantial source of live load for the deck. The larger depth to span ratio yielded a stiffer section, that combined with the higher early average strength of concrete would have resisted tensile stresses in the concrete. Thus, bridge deck 4 exhibited less cracking.

5.11 Conclusions

This portion of the research program examined five bridge decks under construction in Arkansas, the purpose of which was to monitor and document the construction practices used and the performance of the decks. The variables monitored during the five deck placements were compared to the measured crack density. No single variable monitored during the casting of these five bridge decks accurately predicted the likelihood of cracking across the spectrum of bridges. Some variables did show a more general trend with respect to cracking.

Delaying the application of curing leads to increased cracking. The two longest pours in terms of time and volume of concrete did have the most cracking. The bridge with the longest time to final cure application had the most cracking. The bridge with the longest time to initial cure application, curing compound, had the second highest cracking density. Set retarder dosage should be minimized and post-cure mechanical grooving should be investigated as a way to speed the application of protective curing materials. Curing compound application rate should be verified by checking for consistent application pattern and measuring quantity applied.

The evaporation rate the concrete experiences during placement does not control the amount of cracking in the deck. But, coupled with delayed set and delayed curing, it will cause more cracks. Avoiding placements when weather conditions will produce high rates of evaporation would reduce the risk of shrinkage cracks. AHTD should evaluate the effectiveness of overnight placements during the summer construction season.

The use of less flexible girders, more able to resist construction live loads prior to achieving composite action with the concrete slab, could reduce the stresses acting upon the concrete as it sets. The thermal and drying shrinkage stresses in long spans should be investigated. Changes to the nature and location of longitudinal temperature and shrinkage reinforcement should also be considered to alleviate possible transverse drying shrinkage cracks in the positive moment portions of the spans.

Despite some fresh properties measured by the researchers being out of tolerance, all of the contractor and AHTD test values met specifications and all of the procedures and materials used were deemed acceptable under the current AHTD construction specifications. But, some of the material properties and construction procedures documented in this study contributed to the cracking results measured. Clear and direct interpretations of the specifications need to be developed and conveyed to the contractor prior to the placement in order for issues to be resolved.

Monitoring additional bridge decks would strengthen relationships between curing practices and materials and shrinkage cracking in AHTD bridge decks. Future studies should include more measurements of variables investigated in this study to produce clearer records of evaporation rate during the placement and curing. Additional variables that should be monitored include internal and surface concrete temperatures, curing compound application rate and the curing material or concrete moisture state during the seven day cure. Cracking should be measured sooner after placement and the measurements should be repeated on schedule intervals to determine when the cracking begins and at what rate it grows.

Chapter 6

Laboratory Study – Curing Procedures

6.1 Purpose

The purpose of the laboratory study was to test curing procedures in a small scale environment where variables were controlled or monitored. In this way the effects of changes to curing practices could be observed. Various curing methods were applied to small slabs comprised of a standard AHTD bridge deck concrete mixture with the coarse aggregate removed. The use of this paste portion and the proportions of the thin rectangular slabs, as suggested by Kraai (1985) and similar to Shaeles and Hover (1988), should produce a proportional cracking response to that of a full scale concrete bridge deck. The slabs were subjected to cycles of heated air, light and wind to simulate conditions experienced by a concrete deck constructed in the summer construction season. The resulting cracking was measured and the effects of changes in variables are compared based on those results. Additional batches were made to investigate other characteristics of bridge deck concrete, such as time of setting and bleed rate. This work was all completed in the concrete lab at the Engineering Research Center (ERC), Fayetteville, AR.

The chapter begins with a description and discussion of the materials used in the study, followed by details of the slab casting procedure. The description of casting and results of each series of slabs are followed by a discussion of the results for that series. Finally, the conclusions from all of the testing are discussed.

6.2 Materials

6.2.1 Mixtures

The concrete mixture used was taken from two of the bridges previous studied in the project. The mixtures conformed to AHTD construction standards for Class S(AE) Concrete for Structures. Section 802 of the Standard Specifications for Highway Construction specifies that Class S(AE) contain a minimum 611 lb/cu.yd. of cementitious material, the majority being Type I portland cement; although fly ash and slag cement replacements are allowed in limited amounts. The concrete must have a maximum w/cm of 0.44 and a slump of 1 to 4 in. Class S(AE) concrete must be air-entrained to an air content of 6 % plus or minus 2 %, and produce a minimum compressive strength of 4000 psi at 28 days. The concrete mixture proportions and their corresponding Mix IDs are shown in Table 6.2.1.1.

Table 6.2.1.1 Mixture Proportions

Mix ID	w/cm	Cement (lb/yd^3)	Fly Ash (lb/yd^3)	Coarse Agg. (lb/yd^3)	Fine Agg. (lb/yd^3)	Water (lb/yd^3)	AEA (fl oz /cwt)	Set Ret. (fl oz /cwt)
C-1	0.44	519	92	1670	1293	269	0.75	0
R-1	0.44	519	92	1670	1293	269	0.60	2
R-1b	0.44	519	92	1670	1293	269	0.60	2
R-1c	0.44	519	92	1670	1293	269	0.60	2
R-1d	0.44	519	92	1670	1293	269	0.65	2
R-4	0.44	519	92	1670	1293	269	0.85	4
R-8	0.44	519	92	1670	1293	269	0.80	8
CC-1	0.44	519	92	1670	1293	269	0.75	4
CC-2	0.44	519	92	1670	1293	269	0.75	4

The cement used in the laboratory mixtures is Type I portland cement produced by the Lafarge Co, Tulsa, OK, that conforms to AASHTO M 85, Type I. It was obtained and stored in sacks weighing approximately 94 lbs. each. As shown in Table 6.2.1.1, 15%, by weight, of the portland cement was replaced with a Class C fly ash, conforming to AASHTO M 295, obtained from Headwaters Resources, Redfield, AR.

The coarse aggregate was a crushed limestone aggregate stockpiled at the ERC. The source of the aggregate was McClinton-Anchor of Springdale, AR. The aggregate complies with AHTD specifications for soundness, durability, fineness, and deleterious substances and the alternative Gradation AASHTO M43 #57 as shown in Table 6.2.1.2 below. The results of a sieve analysis completed by McClinton Anchor are also included.

Table 6.2.1.2 Coarse Aggregate Gradation

Sieve	Crushed Limestone % Passing	AASHTO M43 #57 % Passing
1-1/2"	100	100
1"	100	95-100
3/4"	74	-
1/2"	35	25-60
3/8"	14	-
# 4	2	0-10
# 8	1	0-5
# 16	1	-

The fine aggregate was washed river sand supplied by Arkhola, Van Buren, AR, and also stockpiled for research at ERC. The fine aggregate complies with AHTD specifications regarding the amounts of deleterious material and the gradation shown in Table 6.2.1.3 below. The results of the sieve analysis are also shown, and were taken from other research using the same stockpile.

Table 6.2.1.3 Fine Aggregate Gradation

Sieve	Fine Aggregate % Passing	AHTD Specifications % Passing
3/8"	100	100
# 4	98	95-100
# 8	92	70-95
# 16	80	45-85
# 30	58	20-65
# 50	18	5-30
# 100	2	0-5

The admixtures were an air-entraining agent conforming to ASTM C 260 and AASHTO M 154, and a set-retarding admixture conforming to ASTM, C 494, Type D and AASHTO M194, Type D. The air-entraining agent used was Daravair 1000 and the set-retarding agent was Daratard 17. These products are manufactured by Grace Construction Products, Cambridge, MA. Both were used in amounts within the manufacturer's suggested dosage and according to the manufacturer's directions. Both are also currently listed on AHTD's Qualified Products List (QPL) and commonly used for AHTD construction projects.

6.2.2 Curing Compound Materials

The curing compound used for these tests was a clear, water-based, membrane-forming curing compound with a fugitive pink dye to aid application. The product used was 1100 Clear Series produced by W. R. Meadows, Hampshire, IL. It conforms to ASTM C 309 Type I and AASHTO M 148, Type I, and is on the AHTD QPL. The compound was agitated prior to application and applied using a one gallon garden sprayer. The plastic sheeting was a 4 mil polyethylene sheeting that conforms to AASHTO M171. The burlap was a cotton burlap of unknown weight.

6.2.3 Formwork

The slabs were cast in 2 foot by 3 foot plywood forms ¾ inches deep. The bottom of the form was lined with 4 mil polyethylene sheeting to prevent water loss through the bottom or sides of the formwork. A 1 ½ inch strip of welded wire reinforcement, with ½ inch square openings, was stapled to the bottom along inside perimeter of the form, and then bent up at a 45 degree angle. See Figure 6.2.3.1 below for details.

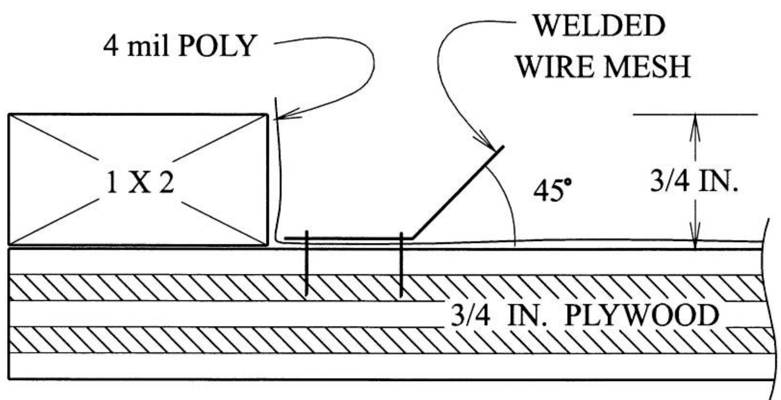

Figure 6.2.3.1 Partial Section of Slab Formwork

6.2.4 Curing Environment

The slab samples cast in batches R-4, R-8, CC-1, and CC-2 were exposed to controlled environmental conditions by placing them inside an insulated chamber assembled for this purpose inside the ERC concrete lab. This chamber was constructed using ¾ inch thick Styrofoam insulation board supported by a wooden framework to create walls and a ceiling. Inside the chamber an electric heater was used to increase the air temperature and four electric box fans were used to simulate wind over each sample. The temperature and relative humidity were measured using a VWR Traceable® Humidity Monitor/Air Thermometer/Clock, manufactured by VWR International, West

Chester, PA. Measurements were recorded periodically throughout the testing. Air speed was measured initially using the wind speed measurement function on the Brunton ADC-PRO weathermeter and assumed to be constant for the same fan arrangement and settings.

6.3 Batching

The component materials for each batch were weighed in plastic buckets prior to mixing. The required weights of cement and fly ash were placed in individual buckets. Aggregates were weighed in to the required number of buckets, each weighing 50 pounds and an additional bucket contained the remainder of the required weight. Lids were then placed on the buckets to prevent a change in moisture content prior to batching. Samples were taken from the aggregate stockpile at the same time, then weighed, and then oven-dried until a constant weight was obtained. Prior to mixing, the moisture content of the materials was calculated and the weight of aggregates and required mix water were adjusted for the water present or absent in the aggregates. Water was measured by weight into a bucket, as well.

The concrete was mixed in a nine cubic foot capacity, electric, revolving-drum mixer. The mixing procedure consisted of first adding the coarse aggregate, approximately half the mix-water, and the air-entraining agent to the mixer. The mixer was turned on and the fine aggregate, cement, fly ash, the remaining water, and the set-retarding agent, if used, were added to the mixer. The concrete mixture was mixed for three minutes, rested for three minutes with the mixer off, and then mixed for two additional minutes prior to discharge, except as noted in the discussion of results for each batch. The mixing procedure followed the procedure outlined in ASTM C 172. Batch sizes ranged from 4 to 7.5 cubic feet depending on test requirements.

6.4 Fresh tests

Prior to the casting of the slabs, the concrete was subjected to fresh property testing. The slump (AASHTO T 119), air content (AASHTO T152), unit weight (AASHTO T 121), and concrete temperature (AASHTO T 309) were measured and recorded. Cylinders, 4 inch by 8 inch, were cast for compressive strength testing at 1, 7, and 28 days, except as noted. These were cured in an environmental chamber at 72 degrees Fahrenheit and approximately 50 percent relative humidity for 24 to 72 hours until they could be de-molded without significant damage. After de-molding, the cylinders were placed in a lime-water bath inside the environmental chamber until testing. Bleed-rate samples were also cast for each batch, using a 0.5 cubic foot unit weight bucket as described in AASHTO T 158 (ASTM C 232). The bleed-rate samples were placed in the environmental chamber described above for the duration of the tests. For batches C-1, R-1b, and R-1c, time of setting was also measured according to AASHTO T 177 (ASTM C 403).

6.5 Casting Procedure

6.5.1 Wet-Sieving

Concrete used for measuring time of set or used for casting the test slabs was wet-sieved to remove the coarse aggregate. The concrete was placed on a standard No. 4 sieve resting over a water-tight container. The concrete was vibrated using an electric internal vibrator until the majority of the paste portion passed through the screen and only the coarse aggregate thinly coated with paste remained. The paste-coated coarse aggregate was discarded and the process was repeated until a sufficient amount of concrete paste was obtained.

6.5.2 Casting

The concrete for the test slabs for batches R-4, R-8, CC-1, and CC-2 was placed in the plywood forms described above and consolidated using the edge of a steel trowel. Attention was given to fully consolidating the concrete around the edges of the form and the wire reinforcement. The concrete was then struck off using a piece of steel angle (1 inch by 1 inch by 1/8 inch and approximately thirty-six inches long) as a screed. The screed was slowly advanced along the 3 foot length of the form (the longitudinal direction) with a slight side to side motion parallel to the short side of the form (the transverse direction). The surface was finished by the minimum transverse passes of the steel trowel required to produce a smooth, uniform appearance. Portions of some slabs were tined using a twelve inch tine rake constructed of 3/16 inch wide steel tines spaced 3/4 of an inch on center and attached to a wooden block. The time of placement, screeding, finishing, and tining was recorded for each slab cast. Samples of the paste were also tested for flow using ASTM C 230 (No AASHTO equivalent) as slab casting commenced and when casting was completed.


Figure 6.5.1 Slabs Cast for Batch CC-2 in Curing Chamber
Note: The front wall of the curing chamber is not in place.

6.5.3 *Curing*

When the finishing and tining of the surface was complete, the slabs were arranged in the testing chamber and exposed to simulated wind to allow any bleed water to rise and be evaporated prior to curing compound application. The exact timing was dependant on the disappearance of the free water from the surface of the slabs and is reported in the results section for each test batch. Curing compound was applied using a manual pump garden sprayer. The sprayer was filled with the compound to the recommended fill line and then weighed on digital scale. The sprayer was then pumped fifteen times to achieve an adequate pressure. Moving the sprayer in one direction only, the compound was applied to one test slab at a time, being careful to avoid loss to overspray and achieve a uniform distribution. The sprayer was then weighed again. The difference between the weights was compared to pre-calculated target weights based on half of the desired total application rate for that sample. Additional compound was applied to the slab until the desired weight of liquid had been applied. This process was then repeated for each slab in the test receiving curing compound. The samples were then exposed to an additional period of wind and, or heat prior to the second application of curing compound. The second half portion of compound was applied with the same procedure as the first, but perpendicular to the direction of the first application. After an additional period of drying, samples also receiving wet burlap were covered.

The completed test slabs were then closed in the curing chamber and subjected to approximately six hours of increased heat and airflow. After this exposure period each day, the heater and fans were shut off and the chamber left sealed to allow the temperature to return to ambient lab conditions. This was to simulate the daily cycles of heat and wind that a bridge slab might experience in its environment. This cycle was repeated for three to seven days after casting as recorded in the results section for each test. The environmental conditions were periodically recorded as described above. Burlap coverings were wetted periodically to prevent the wicking of moisture from the slabs. Samples with burlap coverings were also covered with sheets of 4 mil polyethylene to reduce the required re-wettings, similar to curing practices observed in the field study.

The environmental conditions of the chamber were used to calculate an evaporation rate using an equation published by Paul Uno (1998). The equation provided a direct calculation method for evaporation rate instead of the ACI 305R Nomograph based on equations by Menzel. The equation uses simple input values in units easily attainable using a device similar to that used in the field study. The equation and input variables are as follows:

$$E = (T_c^{2.5} - r * T_a^{2.5})(1 + 0.4\ V) \times 10^{-6}$$

Where:

E = Evaporation rate, lb/ft^2/hr,

T_c = Surface temperature of the concrete, °F,

r = Relative Humidity, (% / 100),

T_a = Air temperature, °F, and

V = Wind velocity, mph.

A resulting graph of evaporation rate versus time for a representative curing cycle is shown in Figure 6.5.2, along with the temperature and relative humidity data.

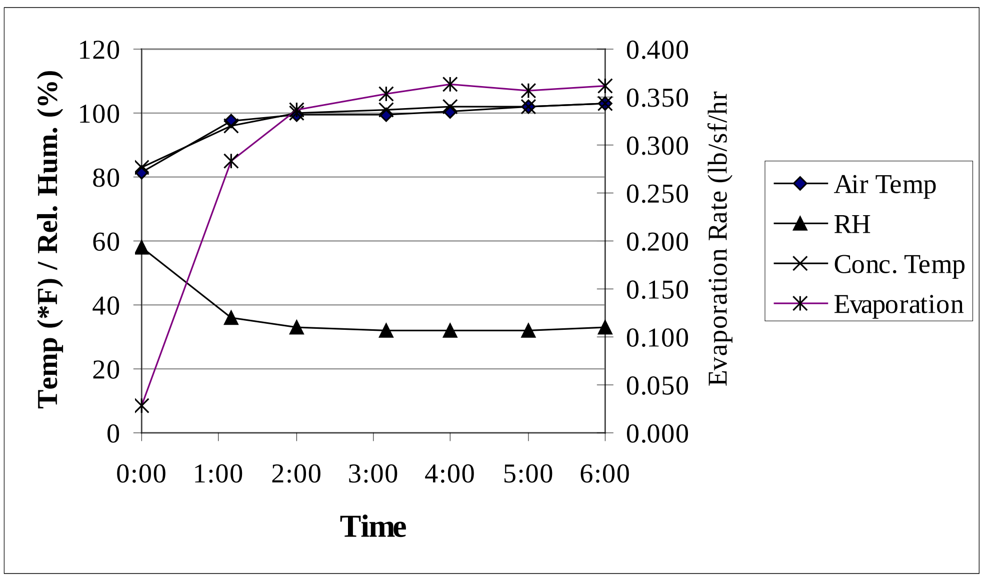

Figure 6.5.2 Typical Curing Conditions

6.6 Cracking Measurement

At approximately 1-day of age, the cracking in each slab was measured and recorded. This was done by locating any cracking present and marking them with a felt tip marker. Subsequent growth of the cracks could then be observed by inspecting the marked locations. The locations of cracks and the extents of tined surface were mapped onto a prepared form for each slab. The widths of the cracks were measured with the crack comparator card, and representative widths were recorded for portions of cracks on the forms, along with the date and time of mapping. Digital photographs were also taken to document the overall appearance of the slab. At the completion of testing, on the third or seventh day, depending on the test, this mapping process was repeated for each test slab using the same form as for the first day. Changes to crack width or length were then quickly noted on the original maps. A final photo was also taken of each slab prior to disposal.

The resulting crack maps were then scaled and the values of length and width entered in to a MS Excel (Microsoft) worksheet for analysis. The cracks were also coded for their orientation (longitudinal, transverse, or angled) and whether they were in the tined portion of the test slab. Tabulations could then be made for length of crack, area of cracking, and average width of crack per slab referenced to mapping time, orientation, or whether in the tined portion or not. These are shown in the results section for each of the 4 slab tests.

6.7 Preliminary Batch Results

6.7.1 Batch C-1 Control Mixture

The initial step in this laboratory study was to batch a control mixture, a mixture proven to meet AHTD bridge deck concrete specifications, without the set-retarding admixture, to determine the baseline characteristics of the concrete. The mix proportion (Mixture C-1) shown in Table 6.2.1.1 was used to batch approximately 4.4 cubic feet. The fresh property values are shown in Table 6.7.1.1 below, as well as the average compressive strength values for 7 and 28 days. No 24 hour compressive strength tests were made.

Table 6.7.1.1 Fresh Properties and Compressive Strengths for C-1

Test	Mixture C-1	AHTD Spec Range
Slump	5.5 in.	1 in. – 4 in.
Air Content	6.8 %	6 % + or – 2%
Unit Weight	142.16	142.3 (Calculated Target)
Concrete Temperature	62 °F	50 °F – 95 °F
Compressive Strength		
7 – Day	3380 psi	No Minimum
28 – Day (+ 5 days)	5130 psi	4000 psi

Bleed rate samples were monitored for approximately 4 hours and fifteen minutes in the environmental chamber, at which time they were weighed again. Three samples were cast to measure bleed rate. Some difficulties had been encountered in collecting the water from the surface of the samples, and as a result some bleed water was lost. To evaluate the impact of this on the final numbers, the bleed rate was calculated twice for each sample, using the volume of water collected from the surface of each sample and using the difference in sample weight from the initial to final measurement. The results from these tests are shown in Table 6.7.1.2.

Table 6.7.1.2 Bleed Rate Values for Batch C-1

Bleed Rate (lb/sf/hr)	Sample 1	Sample 2	Sample 3	Average
By Water Volume	0.0236	0.0157	0.0157	0.0183
By Weight Difference	0.0294	0.0216	0.0233	0.0248

The values calculated from the collected water volume for Samples 2 and 3 are lower than the others. These samples were subject to the most losses. Thus, the average value of 0.0248 lbs/ sq. ft./hr calculated from the weight differences best represents the bleedrate for this sample. This is much lower than the widely reported 0.20 lbs/sf/hr considered to be the plastic shrinkage threshold, when evaporation rate exceeds the bleedrate of the concrete.

Using the test methods of AASHTO T 197 (ASTM 403) the time of set for C-1 was also measured in the environmental chamber. C-1 was found to reach initial set in 7 hours and 35 minutes and final set in 9 hours and 41 minutes.

6.7.2 Batch R-1 Set-Retarded Mixture

To measure the effects of set retarding agent on concrete properties, the same battery of tests described above were repeated for R-1. The only variations between the batches was the addition of set-retarder, at the rate of 2 fl oz/ cwt, and the small reduction in the AEA dosage, to account for the air-entraining properties of the set-retarder. Approximately 4.8 cubic feet of concrete were made for these tests. The fresh property values are shown in Table 6.7.2.1 below, as well as the average compressive strength values for 7 and 28 days. No 24 hour compressive strength tests were made.

Table 6.7.2.1 Fresh Properties and Compressive Strengths for R-1

Test	Mixture R-1	AHTD Spec Range
Slump	5.75 in,	1 in. – 4 in.
Air Content	5.75 %	6 % + or – 2%
Unit Weight	143.4	142.3 (Calculated Target)
Concrete Temperature	56 °F	50 °F – 95 °F
Compressive Strength		
7 – Day	4550 psi	No Minimum
28 – Day (+1 Day)	5830 psi	4000 psi

Bleedrate samples were monitored for approximately 8 hours in the environmental chamber. No collection losses were experienced, but the bleedrates were calculated in the same two ways as for batch C-1 for the purpose of comparisons. The bleedrate results for batch R-1 are given in Table 6.7.2.2 below. Each method's values lie close to the average for that method, but no experimental data indicates the cause of the difference between values for the two methods of calculating the bleedrate.

Table 6.7.2.2 Bleedrate for Batch R-1

Bleed Rate (lb/sf/hr)	Sample 1	Sample 2	Sample 3	Average
By Water Volume	0.0249	0.0251	0.0292	0.0264
By Weight Difference	0.0188	0.0184	0.0161	0.0178

Time of set samples were also prepared for batch R-1 and placed in the environmental chamber. However, 10 hours after batching, the samples had only reached an average resistance value of 22.5 psi (Initial Set corresponds to a penetration resistance of 500 psi). The samples were then left overnight to be tested again the following morning, at which time they had exceeded the final set value.

6.7.3 Batch R-1b Time of Set for Set –Retarded Mixture.

Using the same mixture proportioning and dosage rates as Batch R-1, Batch R-1b was cast for the purpose of measuring the time of set for the set-retarded concrete. The same group of fresh properties was measured. The results are shown below, along with the compressive strength test results, in Table 6.7.3.1. The batch size was 2.2 cubic feet, but only yielded enough for three time of set samples.

Table 6.7.3.1 Fresh Properties and Compressive Strengths for R-1b

Test	Mixture R-1b	AHTD Spec Range
Slump	5.0 in,	1 in. – 4 in.
Air Content	6.75 %	6 % + or – 2%
Unit Weight	143.4 pcf	142.3 pcf (Calc'd Target)
Concrete Temperature	52 °F	50 °F – 95 °F
Compressive Strength		
7 – Day	4460 psi	No Minimum
28 – Day (+1 day)	5470 psi	4000 psi

The average initial set time for the R-1b samples was calculated as 20 hours 11 minutes, with final set at 21 hours 55 minutes. This more than 100 percent increase in set time versus the C-1 batch (Control batch) can not be wholly attributed to the set-retarding agent, which was added at the manufacturer's minimum recommended dose. The air temperature was approximately 40 °F at the time the aggregates were sampled from exterior stockpiles for this test batch. The low material temperature resulted in a low concrete temperature, which likely contributed to the increase in set time.

6.7.4 Batch R-1c Time of Set for Set-Retarded Mixture in a Heated Environment

To evaluate the effects of the same set-retarder dosage, 2 fl oz /cwt, in the hotter, drier conditions likely to be experienced in the summer construction season, R-1c was batched using the same mixture proportions as the previous set-retarded mixtures. Time of set was then measured for samples exposed to heated air and simulated wind inside a plastic enclosure in the concrete lab. The enclosure for this test was constructed of plastic supported on metal frames. The heat was provided by two electric space-heaters and the simulated wind was made by small, electric fans. To overcome the low temperatures of the stockpiled aggregates, the aggregates were sampled on the previous day and allowed to come to lab temperature overnight. Heated water, approximately 125 °F, was also used in mixing the concrete.

Prior to the time of set test, the fresh properties were measured and recorded and cylinders were cast for compressive strength testing. The results of these tests are shown in Table 6.7.4.1. This batch was increased from 2.2 to 3 cubic feet of concrete to allow for 4 time of set samples. However, the lower slump value reduced the yield of the wet-sieving procedure, and thus, only three samples were made. The time of set samples were exposed to approximately eight hours of increased air temperatures, 83.5 °F to 95.0 °F, with relative humidity ranging from 44 to 35%, and wind speeds of 5 – 8 mph. Average time of initial set for the samples of R-1c was found to be 6 hours and 42 minutes, and final set was determined to occur at an average of 8 hours and 7 minutes.

Table 6.7.4.1 Fresh Properties and Compressive Strengths for R-1c

Test	Mixture R-1c	AHTD Spec Range
Slump	1 3/4 in,	1 in. – 4 in.
Air Content	4.9 %	6 % + or – 2%
Unit Weight	145.7 pcf	142.3 pcf (Calc'd Target)
Concrete Temperature	82 °F	50 °F – 95 °F
Compressive Strength		
7 – Day	4980 psi	No Minimum
28 – Day (+1 Day)	6280 psi	4000 psi

6.8 Slab Study Batch Results

6.8.1 Batch R-4 Thin Slab Cracking Test – Three Curing Methods

The first test series evaluated three sample slabs, each given a different curing method, versus a control sample. The concrete mixture included the set retarding agent at a dosage rate of 4 fl oz/cwt. The slabs were cast as described above. Slab A, the control, was finished smooth (not tined) and was not given any additional moisture or protective covering. Slab B was partially tined and covered with wet burlap. Slab C was also partially tined and covered with curing compound at a target rate of 125 sf/gal. Slab D was partially tined, coated with curing compound at the same target rate, and covered with wet burlap. The difference in measured cracking should indicate the effectiveness of the curing regimens.

6.8.1.1 **Fresh Properties for Batch R-4.** The results of fresh and hardened property tests for the batch are shown below in Table 6.8.1.1. The batch size for test R-4 was 7.5 cubic feet. The lower than expected unit weight and higher slump could be attributed to the larger dose of AEA added to achieve the target air content of six percent. Results of the flow table test (ASTM C 230) are also included in Table 6.8.1.2 for three tests taken during the casting process. These results show the mix had low workability when compared to the mortars used in tests by Shaeles and Hover (1988) and the flow diminished during casting. Bleed rate samples were also cast and placed in the environmental chamber, but no measurable bleed water was collected in 6 hours of monitoring.

Table 6.8.1.1 Fresh Properties and Compressive Strengths for R-4

Test	Mixture R-4	AHTD Spec Range
Slump	6 in,	1 in. – 4 in.
Air Content	9.0 %	6 % + or – 2%
Unit Weight	136.6 pcf	142.3 pcf (Calc'd Target)
Concrete Temperature	72 °F	50 °F – 95 °F
Compressive Strength		
1 – Day	400 psi	No Minimum
7 – Day	4130 psi	No Minimum
28 – Day	5420 psi	4000 psi

Table 6.8.1.2 Flow Measurements for R-4

Test Time	Elapsed Time	Spread %
12:03 PM	33 min.	103
12:30 PM	1 hr.	92
1:10 PM	1 hr. 40 min.	86

6.8.1.2 **Casting and Curing for Batch R-4**. The paste fraction for the four test slabs was placed between 45 minutes and one hour and 30 minutes after batching. The individual slabs were screeded and finished within 15 minutes of their placement. Transverse tining was applied to a strip approximately 12 inches wide at the near end of slabs B, C, and D within five minutes of finishing. The first portion of curing compound was applied to slabs C and D after approximately 30 minutes. At this time, two hours and 13 minutes after batching began; the fans were turned on and directed at the surface of the slabs. The second portion of curing compound was applied one hour after the fans were started, when the first application had dried. After the second application of curing compound to slabs C and D, the heater was turned on and the curing chamber was closed to allow the air temperature to increase. When the second application of curing compound dried, slabs B and D were covered with wet burlap; three hours and 47 minutes after batching began.

The test slabs of batch R-4 were cured in the chamber for seven days and subjected to seven cycles of increased temperature and air flow. The air temperature and relative humidity were recorded periodically, and the burlap was re-wetted to prevent it from drying out. The frequent need for rewetting in the first cycle was reduced by covering the wet burlap with the polyethylene. Average values for air temperature and relative humidity for each of the seven cycles are shown in Table 6.8.1.3 below. The average evaporation rates shown in Table 6.8.1.3 were calculated from the previously described evaporation rate equation published by Uno (1998). The concrete temperature was not recorded for this test, so for the purpose of these calculations it was assumed to be the same as the air temperature. In later tests, when the concrete surface temperature was recorded, this was found to be a valid assumption when initial concrete temperature was close to laboratory temperature.

Table 6.8.1.3. Curing Conditions for R-4*

Cycle #	1	2	3	4	5	6	7
Avg. Air Temp. (°F)	84.0	88.6	90.1	94.5	95.5	94.6	95.4
Avg Rel. Humidity (%)	35.7	36.8	44.3	43.2	42.8	46.2	46.0
Avg. Evap. Rate (lb/sf/hr)	0.21	0.24	0.23	0.26	0.27	0.24	0.25

*Wind speed was 10 mph for all readings.

6.8.1.3 **Cracking Measurements for Batch R-4**. The cracking on all slabs was measured and recorded at one day and seven days, using the process described previously. The values are reported in Table 6.8.1.4. Slab D showed no visible cracks at one or seven day mapping. The widths reported are weighted average widths calculated as the summation of the individual crack lengths times their widths as recorded divided by the total length for each slab. Figures 6.8.1.1 and 6.8.1.2 illustrate the crack orientation distribution for the slabs at one and seven days, respectively. The bar graphs are based on total crack area of each orientation, calculated as total length of crack for that orientation times the average width for the same orientation.

Table 6.8.1.4 Measured Cracking for Batch R-4

Slab ID	Curing Method	1 - Day Total		7 - Day Total	
		Length (in)	Avg. Width (in)	Length (in)	Avg. Width (in)
R-4-A	Control	107.5	0.0267	117.5	0.0269
R-4-B	Wet Burlap	21.5	0.0201	26.0	0.0182
R-4-C	Curing Cmpd	12.0	0.0197	16.0	0.0152
R-4-D	CC and Burlap	0.0	0.0000	0.0	0.0000

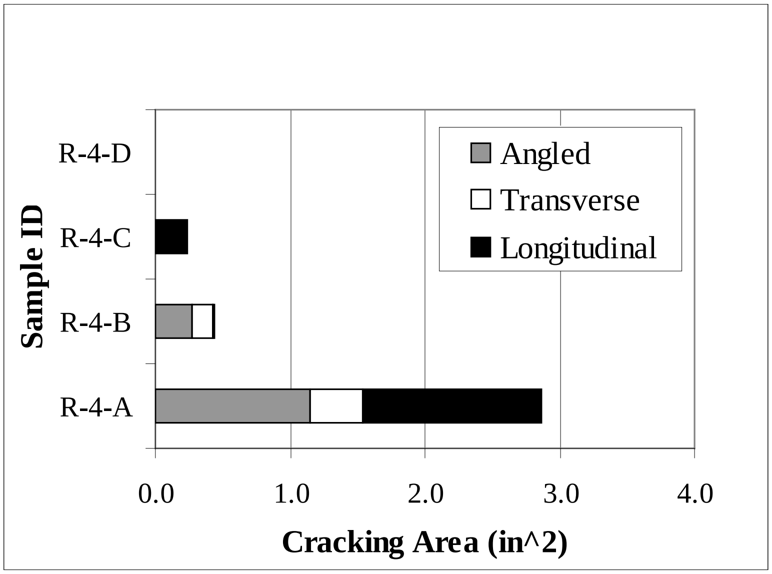

Figure 6.8.1.1 1-Day Cracking Measurements for Batch R-4

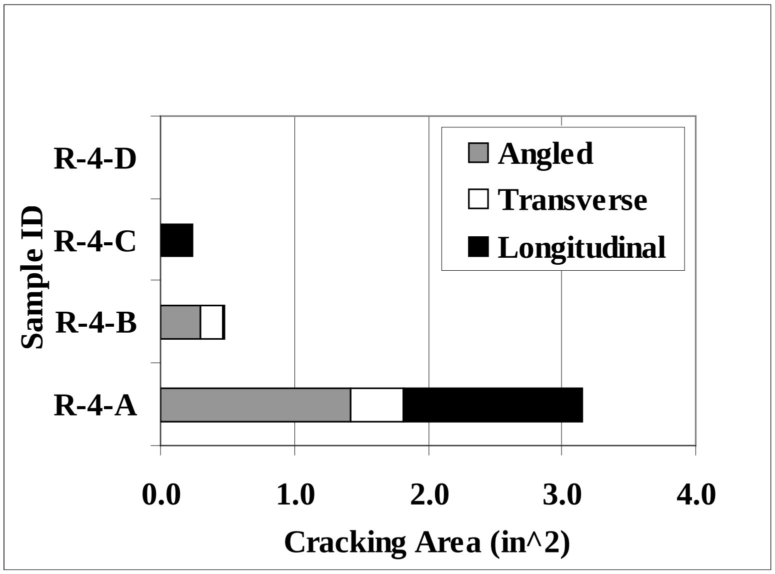

Figure 6.8.1.2 7-Day Cracking Measurements for Batch R-4

Slabs A and B both show an approximate 10% growth in total crack area. Slab A also had a 10% approximate increase in total length, while slab B experienced nearly 20% increase in total length. The orientation distribution does not significantly change from the first mapping to the second. Slab C had less crack length than slabs A and B, but a similar average width at one day. At the second mapping, the total length of cracks on slab C had increased 25%, but the total crack area had only increased 3%. This was due to very small new cracks and only slight growth in existing cracks. As mentioned, slab D, which received curing compound and wet burlap prior to exposure to adverse evaporative conditions did not produce any visible cracking over the entire seven cycles.

6.8.2 Batch R-8 Thin Slab Cracking Test – Increased Set-Retarder Dosage

Batch R-8 repeated the tests of R-4, but with a larger dose of set-retarding agent, 8 fl oz/cwt, compared to 4 fl oz/cwt for Batch R-4. The four slabs were cast and cured in the same manner. Curing compound was applied to slabs C and D at a target rate of 125 sf/gal. Comparisons of measured cracking between R-8 samples should indicate the effectiveness of each curing regimen. Comparisons between R-8 and R-4 could indicate effects of set-retarder dosage.

6.8.2.1 **Fresh Properties for Batch R-8**. The fresh and hardened properties for the batch are shown below in Table 6.8.2.1. The batch size for R-8 was again 7.50 cubic feet. The lower than expected unit weight and higher slump can be attributed to the larger air content. Even though the AEA dosage was reduced from the previous batch, the set-retarding admixture does have mild air-entraining properties. Results of the flow table test are shown in Table 6.8.2.2 for three tests taken during the casting process. These results show similar loss of flow to batch R-4. Bleed rate samples were also cast and placed in the environmental chamber, but no measurable bleed water was collected in approximately five hours of monitoring.

Table 6.8.2.1 Fresh Properties and Compressive Strengths for R-8

Test	Mixture R-8	AHTD Spec Range
Slump	8 in,	1 in. – 4 in.
Air Content	13.0 %	6 % + or – 2%
Unit Weight	130.96 pcf	142.3 pcf (Calc'd Target)
Concrete Temperature	72 °F	50 °F – 95 °F
Compressive Strength		
1 – Day	50 psi	No Minimum
7 – Day	60 psi*	No Minimum
28 – Day	4640 psi	4000 psi

*Average of only 2 samples

Table 6.8.2.2 Flow Measurement R-8

Test Time	Elapsed Time	Spread %
11:05 AM	50 min.	110.4
11:33 AM	1 hr. 18 min.	93.6
12:00 PM	1 hr.. 45 min.	80

6.8.2.2 **Casting and Curing Process for Batch R-8**. The test slabs were placed between 60 and 88 minutes after the batch was taken out of the mixer. The individual slabs were screeded and finished within 15 minutes of their placement. Transverse tining was then applied to a strip approximately 12 inches wide at the near end of the slabs B, C, and D. Approximately two hours after batching began the fans were turned on and directed at the surface of the slabs, the heater was turned on, and wet burlap was applied to slab B. The first portion of curing compound was applied to slabs C and D approximately 10 minutes later. The second portion of curing compound was applied 30 minutes later, when the first application had lost its surface sheen. Wet burlap was added to slab D after the second application had dried. The casting process was completed 2 hours and 45 minutes after casting began.

The test slabs of batch R-8 were cured in the chamber for only three days after casting and subjected to four cycles of increased temperature and air flow. Average values for air temperature and relative humidity for each of the four cycles are shown in Table 6.8.2.3 below. The average evaporation rates shown were calculated as previously described. The concrete temperature was intermittently recorded for this test, so for the purpose of these calculations it was interpolated linearly between measured values. The complete record of temperature and humidity readings can be found in Appendix C.

Table 6.8.2.3 Curing Conditions for R-8

Cycle #	1	2	3	4
Avg. Air Temp. (°F)	93.6	92.5	95.0	93.2
Avg Rel. Humidity (%)	52.1	49.0	49.6	40.7
Avg. Evap. Rate (lb/sf/hr)	0.146	0.224	0.232	0.248

6.8.2.3 **Cracking Measurements for Batch R-8** The cracking on all slabs was measured and recorded at 1-day and 3-days, using the process described previously. The values are reported in Table 6.8.2.4. Slabs C and D showed no visible cracks at one day mapping, and slab D was found to have no cracking on the third day. However, while slab C was initially thought to have no cracks, upon moving the slabs after the final photograph, a large crack appeared. While this may have been caused by flexing the slab, it cannot be proven that crack did not exist prior to movement, so it was documented. Figures 6.8.2.1 and 6.8.2.2 illustrate the crack orientation distribution for the slabs at 1 and 3 days, respectively.

Table 6.8.2.4 Measured Cracking for Batch R-8

Slab ID	Curing Method	1 - Day Total		7 - Day Total	
		Length (in)	Width (in) wtd avg	Length (in)	Width (in) wtd avg
R-8-A	Control	61.0	0.0068	137.0	0.0093
R-8-B	Wet Burlap	29.0	0.0033	39.0	0.0030
R-8-C	Curing Cmpd	0.0	0.0000	21.0*	0.005*
R-8-D	CC and Burlap	0.0	0.0000	0.0	0.0000

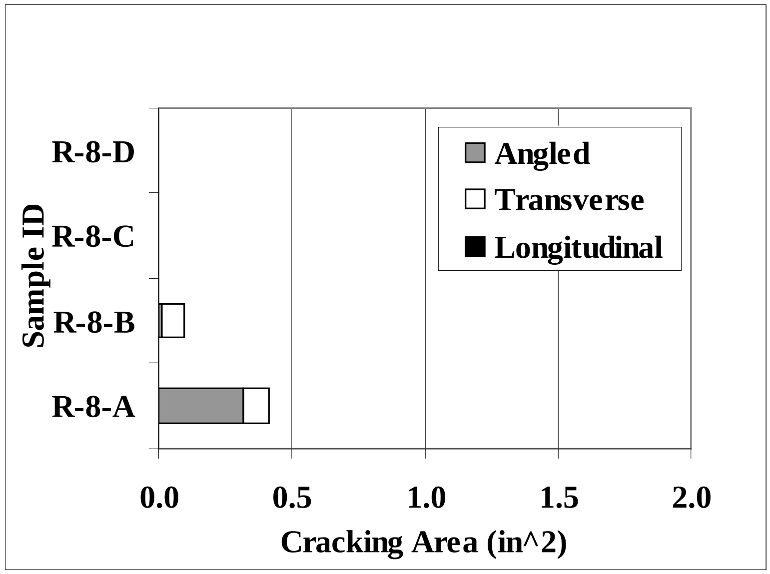

Figure 6.8.2.1 1-Day Cracking Measurements for Batch R-8

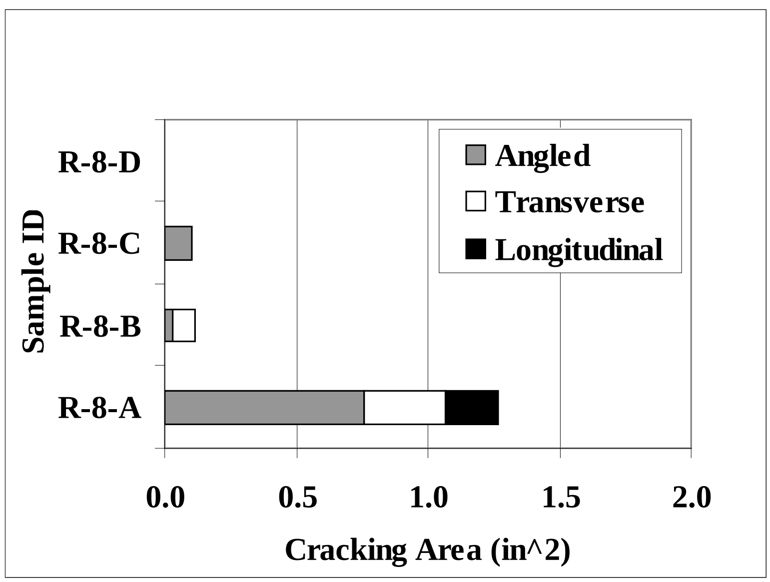

Figure 6.8.2.2 3-Day Cracking Measurements for Batch R-8

Slab A exhibited numerous new cracks visible at the time of mapping on the third day, and previously recorded cracks showed significant growth. Slab A had a 125 percent increase in the total length for cracking and a 37 percent increase in the average width of cracks. This yielded a 207 percent increase in the crack area between the 1-day and 3-day mappings. Slab B cracks only increased 34 percent in length and showed a 10 percent decrease in average width, yielding a 22 percent increase in crack area growth. Cracks recorded on Slab B after one day showed no change after the third day. The loss in average width was due to the presence of new, finer cracks. The proportion of angled to transverse cracks did not change noticeably, but slab A gained additional longitudinal cracks.

6.8.3 CC-1 Variation of Curing Compound Rate

The results of batches R-4 and R-8 indicated that curing compound applied in a timely manner and at an adequate coverage rate could reduce the amount and or size of cracks in the test environment. AHTD construction specifications prescribe a total rate of 125 square feet per gallon of compound. This is more than the manufacturer's recommended minimum of 200 square feet per gallon. To evaluate the effect of this difference and what would happen if this coverage was not achieved, batch CC-1 used four test slabs, three of which received curing compound and one that did not. The three slabs received three different target dosages of curing compound. The AHTD prescribed coverage of 125 sf/gal. was applied to Slab B. The manufacturer's recommended coverage of 200 sf/ gal. was applied to Slab C and Slab D was coated at a rate of only 300 sf/ gal. Slab A served as the control and therefore did receive any curing compound.

6.8.3.1 **Fresh Properties for Batch CC-1.** The batch size for test CC-1 was reduced to 6.5 cubic feet since the number of bleed-rate samples was reduced to one. The results of fresh and hardened property tests for the batch are shown below in Table 6.8.3.1. The slump was one-half inch higher than allowed by AHTD specifications, but the air content was within specifications and the unit weight was very close to the calculated value. Results of the flow table test are also included in Table 6.8.3.2 for two tests taken during the casting process. These results show the mix was very stiff during the casting when compared to previous tests. A single bleed rate sample was also cast and placed in the environmental chamber, but no measurable bleed water was collected in approximately five hours of monitoring.

Table 6.8.3.1 Fresh Properties and Compressive Strengths for CC-1

Test	Mixture CC-1	AHTD Spec Range
Slump	4 1/2 in,	1 in. – 4 in.
Air Content	6.4 %	6 % + or – 2%
Unit Weight	141.60 pcf	142.3 pcf (Calc'd Target)
Concrete Temperature	74 °F	50 °F – 95 °F
Compressive Strength		
1 – Day	320 psi*	No Minimum
7 – Day	4620 psi	No Minimum
28 – Day (+2 Days)	6150 psi	4000 psi

*Average of 1 sample tested at 24 hrs and 2 samples tested at 27 hours

Table 6.8.3.2 Flow Measurement for CC-1

Test Time	Elapsed Time	Spread %
11:26 AM	1 hr. 18 min.	91.8
11:47 AM	1 hr. 39 min.	88.1

6.8.3.2 **Casting and Curing Process for Batch CC-1**. The test slabs for batch CC-1 were placed 60 to 80 minutes after the batch was taken out of the mixer. They were screeded an average of 6 minutes after placement and finished in less than 15 minutes after that. Tining was applied to all slabs, in this test, three minutes after they were finished. Approximately, 100 minutes after the concrete was mixed the fans were turned on and directed at the test slabs. The surface water, evidenced by a pronounced sheen on the surface of the slabs, was then allowed to evaporate until approximately four hours after batching. At this time, although the surface of all slabs still appeared to be wet, a large crack had already begun to form in slab B. The first application of curing compound was made to slabs B, C, and D. Difficulties with the scale used to weigh the applicator prevented the calculation of the amount of compound applied to Slab B. Subsequent applications utilized a different scale. The second application was made after the first had dried for approximately 30 minutes. Casting for batch CC-1 was completed 4.5 hours after commencing the batch.

The test slabs were cured in the chamber for three days after the casting day. In this time they were exposed to four - 6 hour cycles of heat and wind. Average values for air temperature and relative humidity for each of the four cycles are shown in Table 6.8.3.4 below. The average evaporation rates shown were calculated as previously described. The concrete temperature was intermittently recorded for this test, so for the purpose of these calculations it was interpolated between measured values.

Table 6.8.3.4 Curing Conditions for CC-1

Cycle #	1	2	3	4
Avg. Air Temp. (°F)	91.9	89.6	96.2	94.9
Avg Rel. Humidity (%)	45.6	48.5	41.7	38.0
Avg. Evap. Rate (lb/sf/hr)	0.105	0.144	0.255	0.262

6.8.3.3 **Cracking Measurements for Batch CC-1.** After 24 hours, the resulting cracks were located, marked, and mapped as previously described. The crack mapping was repeated at the end of the six hour period on the third day after casting. The values are reported in Table 6.8.3.5. Figures 6.8.3.1 and 6.8.3.2 illustrate the crack orientation distribution for the slabs at one and three days, respectively.

Table 6.8.3.5 Measured Cracking for Batch CC-1

CC-1		1 - Day Total		7 - Day Total	
		Length	Width	Length	Width
Slab ID	Curing Method	(in)	(in) wtd avg	(in)	(in) wtd avg
CC-1-A	Control	46.8	0.0158	51.8	0.0151
CC-1-B	CC - 125sf/gal	24.5	0.0486	25.5	0.0630
CC-1-C	CC - 200sf/gal	9.5	0.0056	14.0	0.0044
CC-1-D	CC - 300 sf/gal	15.0	0.0070	21.5	0.0059

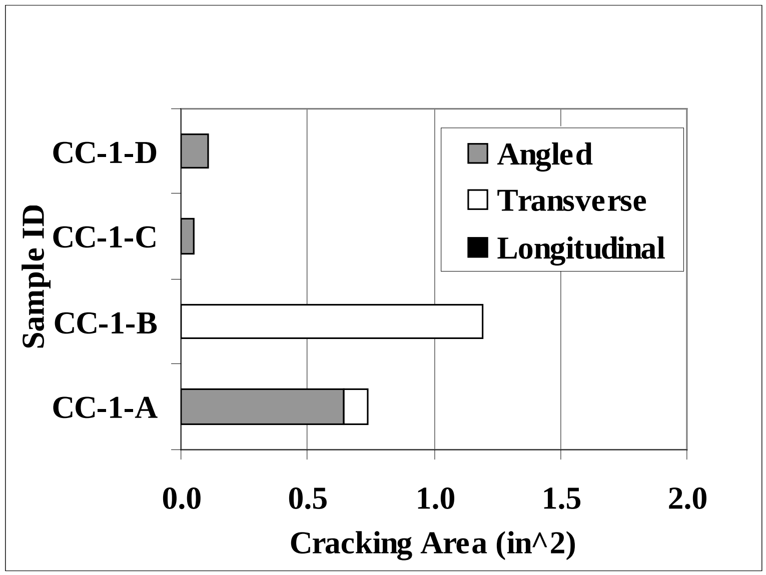

Figure 6.8.3.1 1-Day Cracking Measurements for Batch CC-1

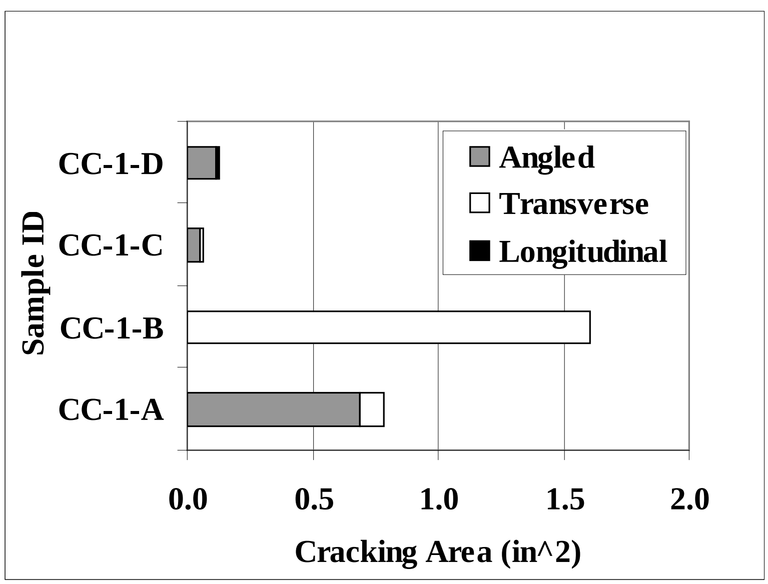

Figure 6.8.3.2 3-Day Cracking Measurements for Batch CC-1

Slab A showed a small, 6 percent, increase in cracking area from one to three days, due to a small increase in length. Slab B, with the most cracking, increased in cracking area 35 percent and average crack width 30 percent. This was not due to any new cracking or increase in length. The middle portion of the large crack that formed prior to curing compound application increased from 0.079 inches to 0.125 inches in width. This crack would cause a significant change in the stress distribution in the slab sample, possibly preventing other cracks from forming or growing. Slab D exhibited more cracking than Slab C. This difference remained proportional over the curing period. Slab D showed 98 percent more cracking area than Slab C at one day of age and 106 percent more at three days of age. The cracking in both was minor compared to Slab A or B.

6.8.4 CC-2 Variation of Curing Compound Rate – Retest

Due to large early crack in and the difficulties in determining the curing compound application rate for slab B in test CC-1, it was decided that the test would be repeated. The mix characteristics and slab treatments were not changed. The batch size was increased to 7.2 cubic feet to accommodate the casting of additional cylinders for other material property research.

6.8.4.1 **Fresh Properties for Batch CC-2.** The results of fresh and hardened property tests for the batch are shown below in Table 6.8.4.1. The slump was lower than the slump of Batch CC-1 and the air content for Batch CC-2 was higher than the air content for Batch CC-1. However, the slump and air content for Batch CC-2 were within AHTD specifications. The unit weight was lower than the calculated value. Results of the flow table test are also included in Table 6.8.4.2 for two tests taken during the casting process. These results show the mix was very stiff. A single bleed rate sample was also cast and placed in the environmental chamber, but no measurable bleed water was collected in approximately six hours of monitoring.

Table 6.8.4.1 Fresh Properties and Compressive Strengths for CC-2

Test	Mixture CC-2	AHTD Spec Range
Slump	3 in,	1 in. – 4 in.
Air Content	7.2 %	6 % + or – 2%
Unit Weight	141.04 pcf	142.3 pcf (Calc'd Target)
Concrete Temperature	72 °F	50 °F – 95 °F
Compressive Strength		
1 – Day	100 psi*	No Minimum
7 – Day	4530 psi	No Minimum
28 – Day	5600 psi	4000 psi

*Average of 1 sample tested at 24 hrs and 2 samples tested at 27 hours

Table 6.8.4.2 Flow Measurement CC-2

Test Time	Elapsed Time	Spread %
11:10 AM	1 hr. 10 min.	72.0
11:24 AM	1 hr. 24 min.	71.5

6.8.4.2 **Casting and Curing Process for Batch CC-2.** The test slabs for batch CC-2 were placed 50 to 75 minutes after the batch was taken out of the mixer. They were screeded within an average of 10 minutes after placement and finished in an average of 22 minutes after that. Tining was applied to all slabs in this test five to nine minutes after they were finished. Approximately 100 minutes after the concrete was batched, the fans were turned on and directed at the test slabs. Thirty minutes later, the first application of curing compound was made to slabs B, C, and D. The curing compound was allowed to dry exposed to the air from the fans and the heater for an additional 30 minutes, and then the second application was made. Two hours and fifty minutes after the batching was completed, the curing chamber was closed to allow the air temperature to increase.

The test slabs were cured in the chamber for three days after the casting day. In this time they were exposed to four - 6 hour cycles of heat and wind. Average values for air temperature and relative humidity for each of the four cycles are shown in Table 6.8.4.3 below. The average evaporation rates shown were calculated as previously described. The concrete temperature was recorded for this test.

Table 6.8.4.3 Curing Conditions for CC-2

Cycle #	1	2	3	4
Avg. Air Temp. (°F)	90.3	92.7	96.3	97.7
Avg Rel. Humidity (%)	38.4	34.1	34.3	36.6
Avg. Evap. Rate (lb/sf/hr)	0.227	0.265	0.298	0.298

6.8.4.3 **Cracking Measurements for Batch CC-2.** After 24 hours, the resulting cracks were located, marked, and mapped as previously described. The crack mapping was repeated at the end of the six hour period on the third day after casting. The values are reported in Table 6.8.4.4. Figures 6.8.4.1 and 6.8.4.2 illustrate the crack orientation distribution for the slabs at one and three days, respectively.

Table 6.8.4.4 Measured Cracking for Batch CC-2

CC-2		1 - Day Total		7 - Day Total	
		Length	Width	Length	Width
Slab ID	Curing Method	(in)	(in) wtd avg	(in)	(in) wtd avg
CC-2-A	Control	32.0	0.0418	32.0	0.0418
CC-2-B	CC - 125sf/gal	19.5	0.0199	19.5	0.0199
CC-2-C	CC - 200sf/gal	8.5	0.0098	8.5	0.0098
CC-2-D	CC - 300 sf/gal	27.0	0.0298	27.0	0.0298

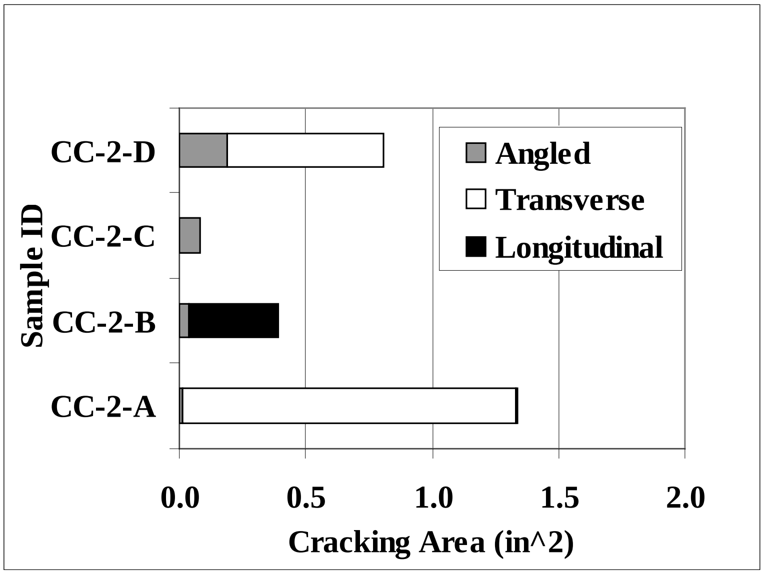

Figure 6.8.4.1 1-Day Cracking Measurements for Batch CC-2

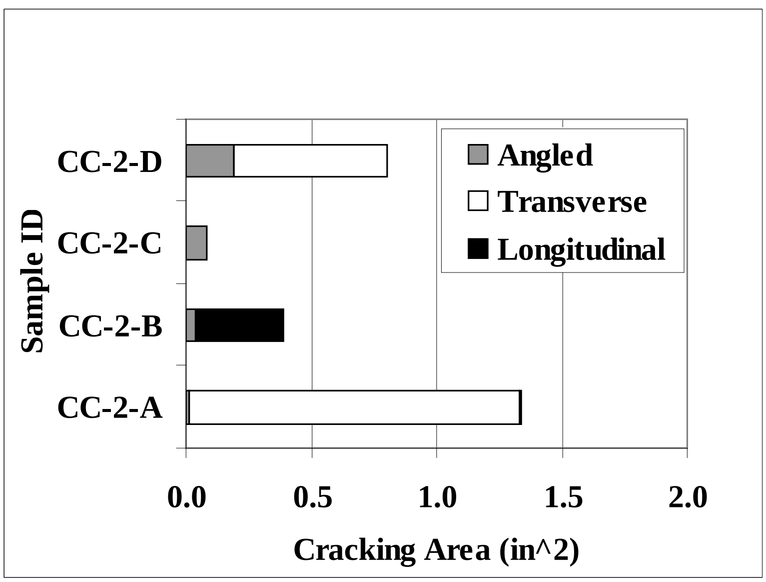

Figure 6.8.4.2 3-Day Cracking Measurements for Batch CC-2

No measurable change in crack length or width was observed in these four samples between the 1-day cracking mapping and the 3-day mapping. All the slabs did exhibit one large crack or a system of interconnected cracks with a few minor cracks in other locations. However, the orientations were not similar. The amount of cracking experienced by each slab does not relate absolutely to the amount of curing compound applied in this test. For example, Slab C received less curing compound than Slab B and still cracked less. But, Slab D received the least curing compound, 300 sf/gal, and had the most cracking. Slab D received less than half of the target amount of compound applied to Slab B, and had twice as much cracking (by area).

6.9 Discussion of Results

6.9.1 Bleeding Rate of Concrete

The bleeding rate of the concrete mixture was measured using the method outlined in AASHTO T 158 for two of the test batches in this study. These batches, C-1 and R-1, produced some measurable bleedwater, averaging 0.0183 lb/sf/hr and 0.0264 lb/sf/hr, respectively. These are the averaged values for three samples for each test and calculated by the volume of collected bleedwater. A single bleed-rate sample was cast for each of the last four tests batches, R-4, R-8, CC-1, CC-2. None of these samples produced a measurable quantity of bleedwater. The slump values for Batches C-1 (5.5 inches) and R-1 (5.75 inches) were higher than allowed by AHTD specification, but slump values for batches R-4 (6 inches) and R-8 (8 inches) were even higher. The air contents for R-4 (9.0 %) and R-8 (13.0 %) were higher than allowed and higher than batches C-1 (6.8 %) and R-1 (5.75 %). The increased air content may have reduced the concrete's ability to settle, and the use of the retarder, which can act as a mild water-reducer, would increase the slump without extra mixing water. Batches CC-1 and CC-2 had slump values (4.5 inches and 3 inches) and air contents (6.4 % and 7.2 %) within AHTD specifications, but would have less bleed water for the same reasons. In the field, the reduced bleed water would mean the concrete had less tolerance for water loss than traditionally thought. Even the measurable rates calculated for Batches C-1 and R-1 were ten times less than the 0.20 lb/sf/hr evaporation rate described as the rate at which evaporation overtakes bleedrate and plastic shrinkage increases.

6.9.2 Time of Set of Concrete Mixtures

Design and Control of Concrete Mixtures (Kosmatka et al., 2002) reports that a concrete mixture with no set-retarding additives with a temperature of 73°F will reach initial set in approximately six hours and final set in approximately nine hours. The time of set calculated for the control mixture, C-1, was 7 hours 35 minutes for initial set and 9 hours 41 minutes for final set. The fresh concrete temperature was 62 °F and the test took place in the environmental chamber at 72 °F. The time of set for the set-retarded mixture R-1b (2 fl oz/cwt of retarder), took 20 hours 11 minutes to reach initial set and 21 hours 55 minutes to reach final set. However, the initial temperature of the mixture was only 52 °F. Mixture R-1C, (2 fl oz/cwt of retarder) achieved initial set in 6 hours 42 minutes and final set in 8 hours 7 minutes, but its initial temperature was 82 °F and it was kept in a much warmer and drier environment. Thus, time of set may follow the expected patterns based on temperature and set retarder dosage, but the time of set range can be quite large. The time of set of AHTD bridge deck concrete should be measured in test batches prior to acceptance of the mixture design to anticipate its effects on finishing and curing operations.

6.9.3 *Curing Compound Application*

Curing compound application rate can be difficult to control. In attempting to attain even and thorough coverage with imprecise equipment, the slabs that received curing compound all received over their target amounts. Summarized in Table 6.9.1, the applications averaged 11 percent over the target amount. This did not represent a substantial amount of compound in this testing. But, in a real slab placement this would be substantial.

Table 6.9.1 Target vs. Actual Curing Compound Application Rates and Crack Areas

Slab ID	Target Appl. Rate (ft^2/gal.)	Actual Appl. Rate (ft^2/gal.)	Over Application (%)	Total Crack Area (in^2)
R-4C	125	114	8.8	0.24
R-4D[1]	125	114	8.8	0.00
R-8C[2]	125	93	25.6	0.11[2]
R-8D[1]	125	103	17.6	0.00
CC-1B[3]	125	171[3]	NA[3]	1.61[4]
CC-2B	125	120	4.0	0.39
CC-1C	200	183	8.5	0.06
CC-2C	200	182	9.0	0.08
CC-1D	300	282	6.0	0.15
CC-2D	300	269	10.3	0.80

[1] Also received wet burlap, [2] Crack may have been post-test,
[3] Actual rate undeterminable due to scale malfunction
[4] Cracking began prior to curing compound application

Waiting for bleed water to dissipate from the surface of the slabs prior to applying curing compound delays curing. During the casting of the slabs for Batch CC-1, the surface of the slabs had sheen for two hours and thirty minutes after casting. This water on the surface prevented the application of curing compound. During this time a large crack had developed in Slab-B. The slabs were being monitored on a twenty to thirty-minute interval when this occurred. This illustrated how quickly the cracking could begin and the results of delayed curing.

6.9.4 Cracking Results with Respect to Curing

The control specimens exhibited the most cracking in all tests except CC-1, where Slab-B, which was to receive curing compound at a 125 sf/gal rate, cracked prior to the application of the compound. Excluding this sample, the uncured control samples had between 1.67 and 10 times more cracking as samples that received some type of applied curing.

In Batch R-4, the control sample, Slab R-4A, had over six times as much crack area of Slab-R4B, which received a wet burlap cure, in just one day. This ratio was nearly the same after 7 cycles of exposure, even though both slabs experienced some increase in cracking. The Batch R-8 control sample, Slab R-8A, had four times as much crack area at one day as Slab R-8B, which also received a wet burlap cure. After four cycles of exposure, R-8A had ten times the crack area as R-8B. The increase in crack area was due to increases in length and width of existing cracks, as well as the formation of new cracks. Slab R-4B had more cracking than Slab R-8B, even though they both were cured with wet burlap. This is likely due to the time of application of the wet burlap. Slab 4-B did not receive its burlap covering for almost three hours after it was cast, while Slab R-8B was covered with wet burlap approximately one hour after its casting.

For Batch CC-1, neglecting CC-1B, the uncured control, Slab CC-1A, had five times the crack area as the next worse Slab CC-1D, which received curing compound at a rate of only 300 ft^2/gal. The Batch CC-2 control, CC-2A had twice as much cracking as the other slabs in the test. Batch CC-2 slasb also had more cracking than Batch CC-1. Batch CC-2 had a lower slump (3 inches) than CC-1 (4.5 inches) and lower flow table measurements than CC-1 (See Table 6.8.3.2, CC-1 and Table 6.8.4.2, CC-2).

The more timely application of wet burlap cures in batch R-8 reduced the cracking between batches R-4 and R-8, but other variations exist between these batches. Higher slump and air content and lower early strength for R-8, may have also reduced cracking. Additional tests, in which slabs from the same batch would be given the same cure at different times, would show the effect of time of cure application.

In all tests, the slab with the most cracking at one day remained the slab with the most cracking, regardless of any measured increase in cracked area from day one to day three or seven. The measured increases in crack area were due to increases in length and width of existing cracks and the appearance of new cracks. However, in Batch R-8, the slab receiving cures, R-8B, R-8C, and R-8D, not only had less cracking than the uncured sample R-8A, but also had less increase from day one to day three. R-8A had more than 200 percent increase in crack area, while R-8B, which received only wet burlap, had only a 20 percent increase in crack area.

6.9.5 Cracking Results with Respect to Curing Compound

All slabs that received curing compound, with or without wet burlap, experienced less cracking than those that received only a wet burlap cure or no applied cure, with the exception of the aforementioned CC-1B. In tests R-4 and R-8, Slabs R-4C and R-8C received curing compound at a rate of less than 125 ft^2/gal, and Slabs R-4D and R-8D received curing compound at the same rate plus wet burlap coverings for the duration of the cure. Slabs R-4D, R-8C, and R-8D exhibited no cracking at one day. The only cracking recorded in any of these three slabs was on Slab R-8C, which may have been damaged caused by moving the slab at the conclusion of the test. Slab R-4C had 45% less cracking and less crack growth than slabs R-4 A and R-4B.

As to the effects of the rate of curing compound application on measured cracking, the results are less clear. As shown in Table 6.9.1,the results of batch CC-2 show that Slab CC-2D (300 ft^2/gal. target rate), which received less than half the curing compound specified by AHTD (125 ft^2/gal.), had the highest crack area of slabs receiving curing compound in that test. However, Slab CC-1D (282 ft^2/gal. actual) received less curing compound than CC-2D (269 ft^2/gal. actual) and had 76 percent less cracking.

Of all the test slabs receiving curing compound, the slabs with the least cracking were R-4D and R-8 D, which experienced no cracking during the tests. In Batches CC-1 and CC-2 in which the effects of curing compound application rate were compared based on cracking, the least cracking occurred in Slabs CC-1B and CC-2B, which received the manufacturer's recommended dosage. The higher dosage required by AHTD specifications did not yield reduced cracking, although the Slab CC-1B was compromised by testing difficulties.

As also noted, the application rates all exceeded the target rates being tested. This combined with difficulties regulating the test mixtures and procedures may obscure direct relationships between the amount of curing compound and the amount of cracking.

6.9.6 Testing Difficulties and Observations

Difficulties in producing a concrete mixture of consistent characteristics created changes in the testing operations that may have affected the outcome of the tests. Although the same mixture proportions, materials, and admixtures were used for all the batches, differences in air content and slump still existed. Air-entraining agent dosage was reduced after batches R-4 and R-8 because those batches were out of AHTD tolerances. The decrease in air content could be the reason for the loss of slump. Other possible causes are changes in the set-retarder dosage, which, as noted before, acts as a low-range water-reducer, and increased air temperatures during batching. The wet-sieving process was exposed to outside temperatures and sunlight. The slump loss caused the process to take longer and reduced its yield, thus extending the process and the exposure. This produced a mortar mixture that was advanced in age and stiff, making precise control of the slab casting more difficult. Even with the set retarder used, it was difficult to regulate the time between batching the concrete and casting the slabs. Previous research using a similar process used only a mortar mix, so there was no delay due to the sieving procedure. In those tests, a precise time schedule was held for all the batches (Shaeles and Hover, 1988). Precision timing of the processes could have produced tighter control over variables, which would have yielded clearer results.

This testing program (conducted at U. of A. E.R.C.) did not produce the same correlation to direction of screeding as reported in Shaeles and Hover (1988). Those researchers found that cracking was typically parallel to the screed used, regardless of whether it was transverse or longitudinal. The cracking in the slabs cast at ERC did not necessarily parallel the screed direction. The slabs in this research exhibited cracking of all directions, even within the same slab sample. Although, as previously stated, this research used a different mixture and a different reinforcing arrangement than those used by Shaeles and Hover. The formwork and reinforcing for the samples in this testing were more closely patterned after Kraai (1985), who did not report such correlations.

6.10 Conclusions and Recommendations

Current standards for curing materials test only individual components of curing system. A standard test to assess the system as a whole needs is needed, because curing effectiveness is impacted by many variables. The test system must account for as many variables as possible. That being said, the test must be more standardized than the current tests in order to produce clearer changes in outcome due to prescribed changes in those variables.

This slab study showed that curing helped prevent cracking in concrete slabs. In all tests except CC-1 the control specimens exhibited the worst cracking. For test CC-1 the application of curing was delayed, thus exposing all the slabs to an uncured condition. For this test a "cured" slab, CC-1B, had more cracking than the uncured control. The uncured control samples had between 1.67 and 10 times the cracking as samples that received some type of applied curing. Cured samples also experienced less crack growth than the control samples. This was best illustrated in Batch R-8, where the difference was by a factor of ten.

Curing compound, as one of these applied cures, was helpful in reducing the amount of cracking. All slabs that received curing compound, with or without wet burlap, experienced less cracking than those that did not receive curing compound, with the exception of the aforementioned CC-1B. The additional curing compound required by AHTD specifications over the manufacturer's recommended rate did not result in reduced cracking. However, this is based on limited comparisons due to the noted difficulties. Additional testing is recommended to verify this result and investigate the effects of cure application timing on slab cracking.

Although no attempt was made in the field study to measure the actual application rates, the amount of curing compound on the all the laboratory test slabs to achieve the target rates did appear to be significantly more than that used on most of the deck placements observed in the field study. Measuring curing compound application rate in field would be difficult. But, calculating the amount required and pre-qualifying spray equipment that can produce desired results would ensure that curing compound was employed effectively against plastic shrinkage cracking.

Chapter 7

Laboratory Study – Curing Procedures II

7.1 Purpose

The purpose of the second laboratory study was to examine additional curing regimens. The same scale slabs and environmental conditions used in the earlier laboratory study were employed in this second phase. Four curing methods were applied: curing compound (ASTM C309), water-based curing compound (ASTM C309), wet burlaps, and wet burlaps with a rewetting stage at mid test. Once the testing was complete, the amount of cracking was measured for each slab.

7.2 Curing Materials

7.2.1 Curing Compound

The curing compound used was a clear, water-based, membrane-forming compound with a pink dye. The specific product was 100 Clear Series produced by W. R. Meadows, Hampshire, IL. It was applied at three dosage rates 100 sf/gal (AHTD specifications), 200 sf/gal (manufacturer's recommendation), and 300 sf/gal (chosen by the research team). The applied dosage was measured by an equivalent weight of compound per a 6 sf. area of slab to the weight of 1 gallon of the curing compound. Moving the sprayer in one direction only, the compound was applied to one test slab at a time achieving a uniform distribution. This compound will be referred as pink curing compound for the remainder of Chapter 7.

7.2.2 Water-Based Curing Compound

A water-based curing compound, THE CURE™ WCE manufactured by SINAK Corporation, was brought to the attention of the research team. This compound was a clear, water-based, non-toxic material containing no volatile organic compounds. This compound was applied in the same manner as the previous membrane-forming curing compound; however, different dosage rates were applied. The higher and lower dosages applied with this compound follow the manufacturer's recommended coverage amount, which are 650, 725, 800 sf/gal. A midpoint dosage of 725 sf/gal was chosen at our discretion based on the average of both recommended dosages. This compound will be referred as SINAK curing compound for the remainder of Chapter 7.

7.2.3 Burlap

The final curing material used was burlap. Burlap mats were soaked in water, excess water was removed, and then placed on the slabs. The wet burlap was placed on the concrete two ways. One method consisted of applying the wet burlaps and leaving them on the slabs for 6 hours. The second method consisted of applying the wet burlaps on the slabs and spraying them with water once they were completely dry. After the second water application, they were left on the slabs for 6 hours, without rewetting. Figure 7.2.1 shows the placement of a burlap mat.

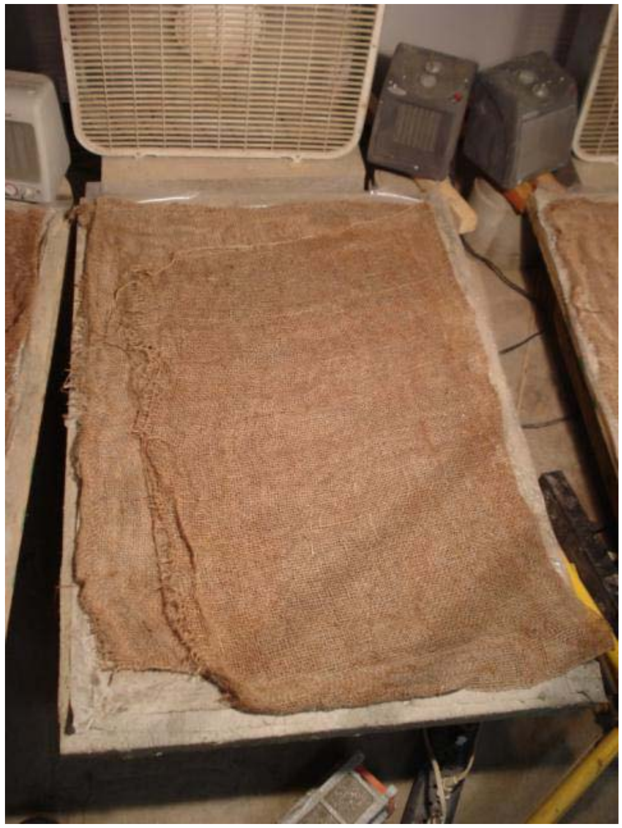
Figure 7.2.1 Slab with Burlap

7.3 Concrete Mixture Proportions

Two different concrete mixture proportions were used in this second phase. The first mixture was similar to that used during Phase 1. This mixture is shown below in Table 7.3.1 and identified as Mixture 1. Due to difficulties with workability and wet-sieving, a second mixture was developed, Mixture 2. This mixture resembled the first, but it contained no coarse aggregate. This mortar mixture allowed the research team to place the slabs without the need to wet-sieve. This mixture is identified as Mixture 2 in Table 7.3.1.

Table 7.3.1 Mixture Proportions

Mix ID	w/cm	Cement (lb/yd^3)	Fly Ash (lb/yd^3)	Coarse Agg. (lb/yd^3)	Fine Agg. (lb/yd^3)	Water (lb/yd^3)	AEA (fl oz /cwt)	Set Ret. (fl oz /cwt)
1	0.44	519	92	1674	1307	250	0.75	8
2	0.44	611	0	0	1521	232	0.00	0

7.4 Concrete Slab Results

7.4.1 Curing Compound – Mixture 1

The pink curing compound manufactured by W.R. Meadows was the first regimen examined. Four slabs were cast twice using Mixture 1 (as shown in Table 7.3.1). The two series are identified as Batch No. 090606 and Batch No. 100406. For each batch, there were four slabs (Slab A, B, C, and D). Slab A was the control slab and did receive the curing compound. Slab B had the highest dosage of curing compound of 125 sf/gal. Slab C had the intermediate dosage of curing compound application rate of 200 sf/gal. Finally, Slab D has the lowest dosage rate of curing compound of 300 sf/gal. For all slabs (except Slab A), the curing compound was applied using a manual pump garden sprayer. The curing compound was applied after all the bleed water had evaporated from the surface of the slabs.

After the slabs were exposed to the heaters, fans, and lights for six hours, there was a waiting period of three days before cracks were measured and recorded. The cracks were inspected visually, marked with a marker, and measured to determine their lengths and widths. The widths of the cracks were measured with the crack comparator card. Each crack was also observed for its direction: longitudinal, transversal or diagonal. Crack area was calculated by multiplying the length of a crack by its width and is shown in Table 7.4.1. The slabs are shown below in Figures 7.4.1 and 7.4.2.

Table 7.4.1 Total Cracking

Batch No.	Area of Cracks (in^2)			
	SLAB A	SLAB B	SLAB C	SLAB D
090606	0.23	0.31	2.46	3.22
100406	1.20	0.80	0.05	1.99

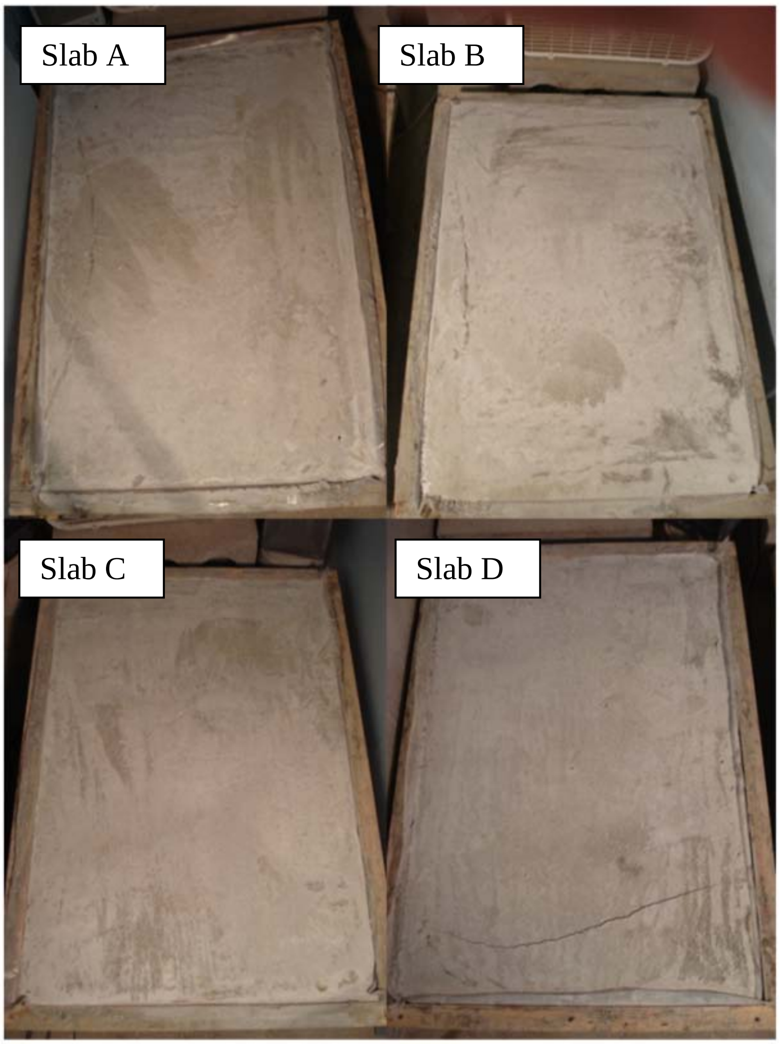

Figure 7.4.1 Slabs A through D for Batch No. 090606

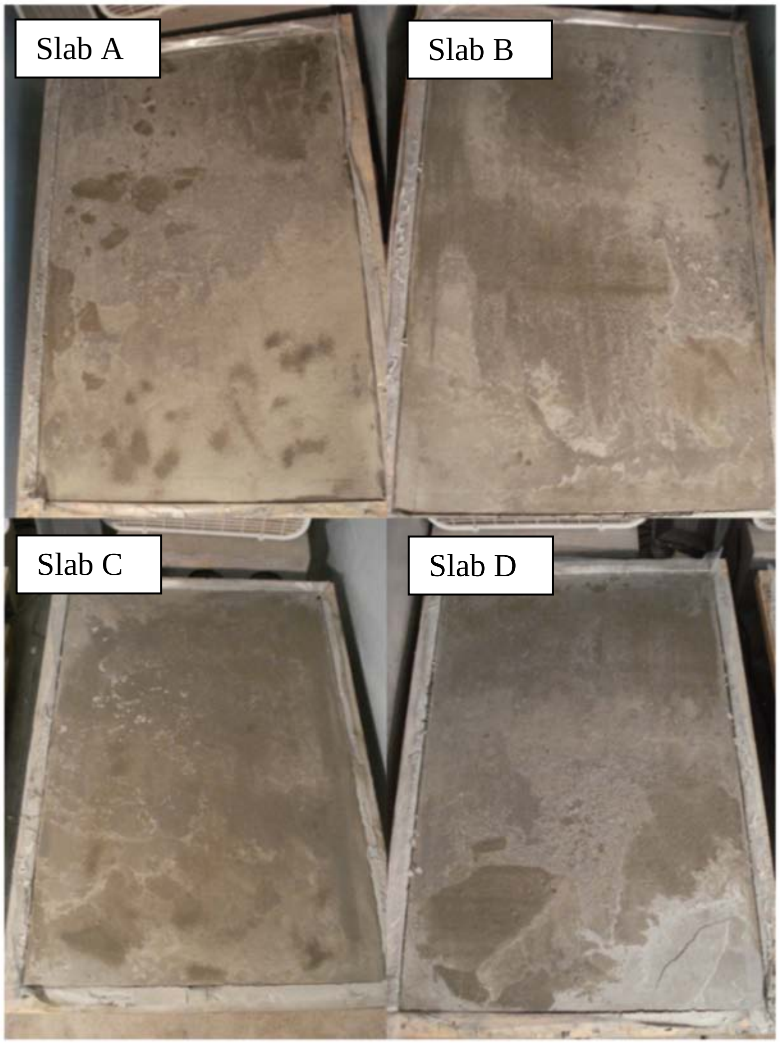

Figure 7.4.2 Slabs A through D for Batch No. 100406

For the two sets of slabs, the highest area of cracks was 3.22 in.^{2} for Slab D of batch 090606 and the lowest was 0.045 in.^{2} for Slab C of Batch No. 100406. The slab with the most consistent area of cracks was Slab B in both batches. This was most likely due to the fact that Slab B had the highest dosage of curing compound; however, Slab B had 0.08 in.^{2} more of cracks than its control slab in Batch No.090606.

As a whole, there were no consistent trends among batches or between batches. For example, it is unclear why Slab C and Slab D developed more cracks than the Control Slab A in Batch 090606. Slab D had a total area of cracks of 3.22 in.^{2} and Slab C had a total area of cracks of 2.46 in.^{2} versus a total area of cracks for Control Slab A of 0.23 in.^{2}. The researchers expected Slab A to have the greatest area of cracks followed by Slabs D, C and B. Since Slab B had the highest dosage rate of curing compound, it was expected to present the smallest area of cracks.

A factor affecting the curing procedure could have been the excessive use of the set retarder. By the time the mortar began hardening, the curing compound had already evaporated or been absorbed by the mortar. Other factor contributing to the inconsistent results include casting methods, finishing, or temperature differences (ambient and concrete); however, a clear reason as to why this occurred is not known.

7.5 Concrete Slab Results - Mixture Proportion No. 2

Because of the previously mentioned disparities in the results, a second concrete mixture was developed. This mixture, Mixture 2, contained no additives and was directly mixed as a mortar in the mixer with no coarse aggregate. However, to design a mortar mix that resembled a concrete mix, coarse aggregate amounts were used for calculating the proportions but were not actually used in the mix.

Trail batches were necessary in order to develop a mortar that had the same consistency and strength as a concrete mixture used in a bridge deck. First, a trial batch was made to test the effectiveness of mixing the ingredients directly as a mortar in the revolving drum mixer. The first mixture was very fluid which was desired to ease placement and to encourage concrete cracking. This first mixture, Mixture 2, (shown in Table 7.4.2) was fluid and achieved a 7 day compressive strength of 4060 psi.

Table 7.4.2 Trial Batch Mix Design

Mix ID	w/cm	Cement (lb/yd^3)	Fly Ash (lb/yd^3)	Coarse Agg. (lb/yd^3)	Fine Agg. (lb/yd^3)	Water (lb/yd^3)	AEA (fl oz /cwt)	Set Ret. (fl oz /cwt)
1	0.44	519	92	1674	1307	250	0.75	8
2	0.44	611	0	0	1521	232	0.00	0

Seven batches were made to test different curing methods using Mixture 2. An initial first batch was created only to test the new SINAK curing compound. This batch was called SINAK testing batch. Four batches were cured using the SINAK compound and two were cured using traditional wet-burlaps. SINAK curing compound was applied as soon as the concrete stopped bleeding and all surface water had evaporated. The initial SINAK testing batch was mainly to test the application of the compound and observe its performance. The results of the batches cured with the SINAK compound will be discussed first, followed by the results of the batches cured with wet-burlaps, and finally the average results of both methods will be compared.

7.5.1 SINAK – Mixture 2

Four sets of slabs were cast to investigate the effects of SINAK on cracking. These four batches are labeled 030107, 032207, 032507, and 041007. Due to additional trial batching and changes in ambient conditions, Mixture 2 was again modified. The mortar proportions used to the cast the slabs are shown below in Table 7.5.1. Shown in Table 7.5.2 are the ambient conditions and mortar strength for the mortar.

Table 7.5.1-1 Mix Design for Batches Cured with SINAK

Batch Weights (yd^3)					
		Batch ID			
		030107	032207	032507	041007
Cement	lbs	585	585	585	585
Rock	lbs	-	-	-	-
Sand	lbs	1311	1302	1320	1319
Water	Lbs	329	338	320	321
AEA	fl oz./cwt	0	0	0	0
Retarder	fl oz./cwt	0	0	0	0

Table 7.5.1-2 Summary of SINAK Results

Batch ID	Ambient Conditions		Compressive Strength (psi) (28 days)
	Max Temperature (°F)	Min Relative Humidity (%)	
030107	102	43	4125
032207	112	40	3900
032507	96	23	3500
041007	98	20	3500

7.5.1.1 **Curing Procedure and Total Crack Results**. The curing compound was applied to the slabs at the same rates for all four batches. For example, Slab A was the control slab for the four mixtures and had no curing compound applied. Slab B had the highest amount of SINAK applied (650 ft^2/gal), Slab C was next with 725 ft^2/gal, and then Slab D, which had the least amount of compound applied (800 ft^2/gal). After the bleed water had evaporated from the surface of the slabs, the SINAK curing compound was applied with a regular household sprayer to each slab, except the control Slab A.

Table 7.5.3 displays the numerical values of the total area of cracks for each slab for the 4 batches. Figures 7.5.3.1 and 7.5.3.2 show each slab for Batch 032507 and Batch 041007, respectively. The highest area of cracks was 4.55 in.^{2} in Slab A of batch 030107 and the lowest area of cracks was Slab B of batch 032207 which did not crack.

Although all four batches were different from one another, there was an apparent trend in three of the batches. In three of the four batches, Slab high exhibited the greatest amount of cracks. This quantity decreased in Slab B and then increased in Slab C and again in Slab D, corresponding to the dosage of curing compound applied to the slabs. Slab A had no curing compound, thus the high area of cracks. Slab B had the highest dosage and resulted in the smallest area of cracks. The SINAK curing compound dosage was less in Slab C and least in Slab D. Each of these slabs exhibited greater cracking. Batch 032507 did not follow this pattern perhaps because there was a misapplication of the curing compound. This trend can be observed in Figure 7.5.1.3.

Table 7.5.1-3 Area of Cracks per Slab

Batch ID	Area of Cracks (in^2)			
	SLAB A	SLAB B	SLAB C	SLAB D
Batch 030107	4.55	2.19	2.72	2.92
Batch 032207	1.09	0.00	0.41	0.49
Batch 032507	1.90	2.37	1.64	2.91
Batch 041007	0.95	0.61	0.81	0.87

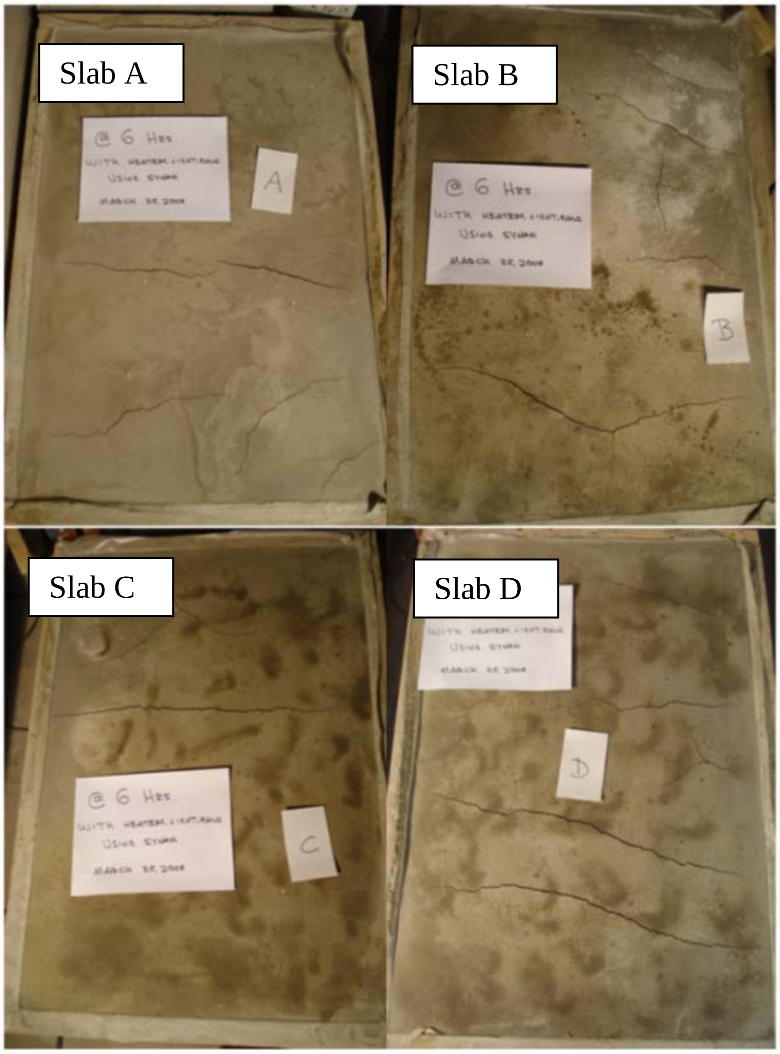

Figure 7.5.1.1 Batch No. 032507

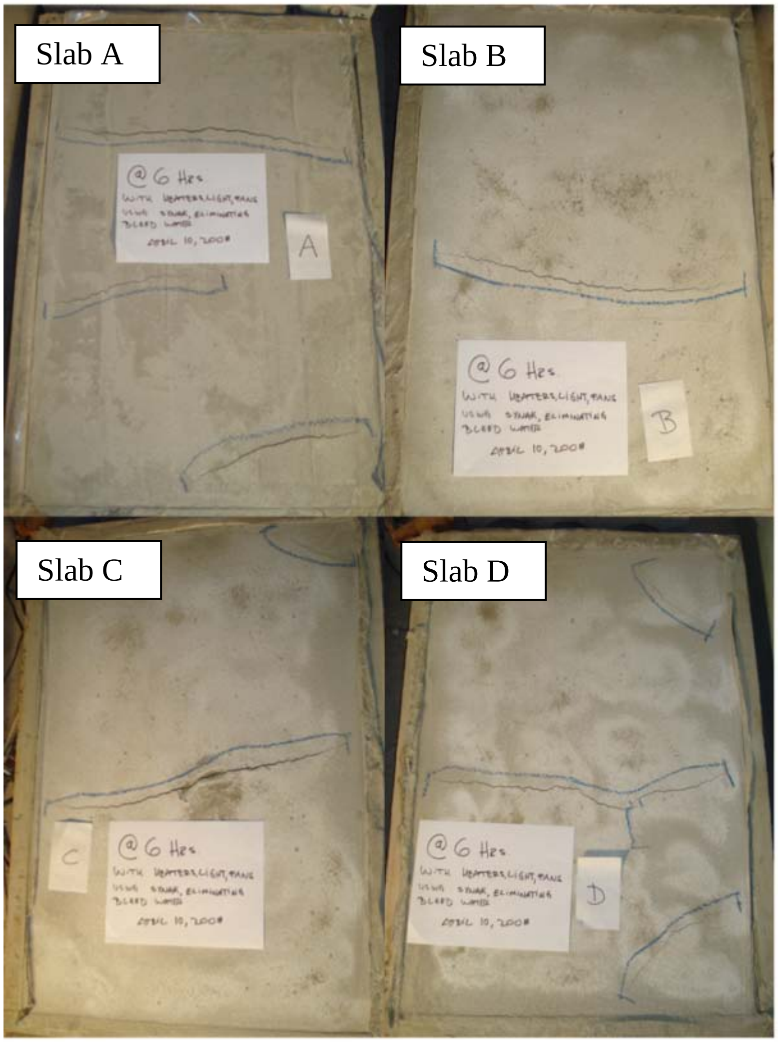

Figure 7.5.1.2 Batch No. 041007

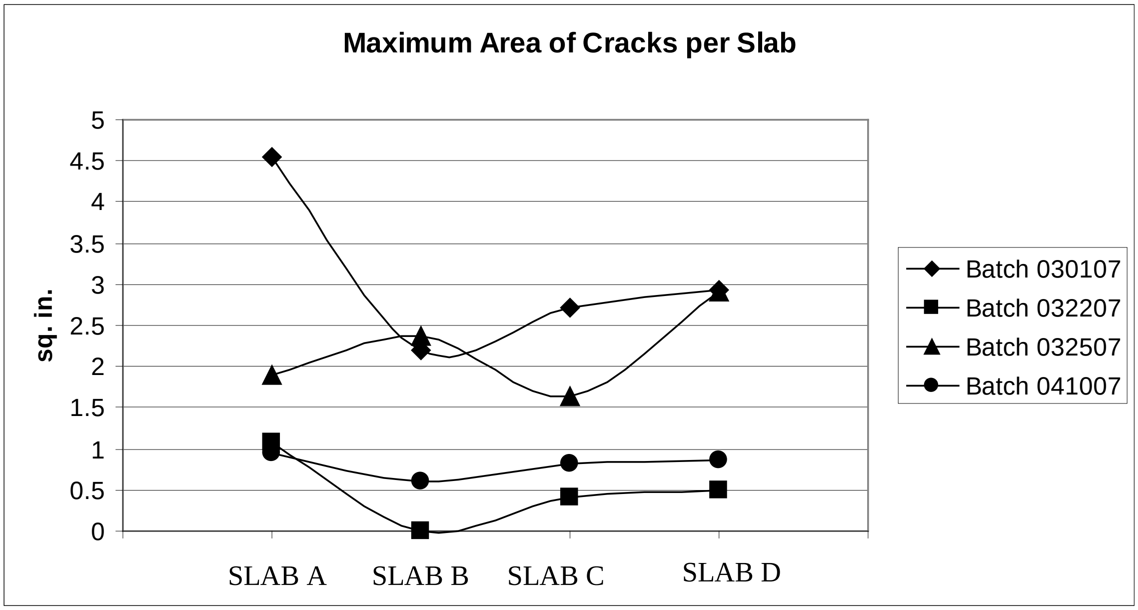

Figure 7.5.1.3 Trend of Maximum Area of Cracks per Slab

7.5.2 Wet Burlap – Mixture 2

Two batches using mixture proportions shown in Table 7.5.2.1 were mixed to compare SINAK with wet-burlap. The method that was used for the first batch, 040308, was wet-burlap placed on the slabs once the bleed water had evaporated. The burlap was not completely soaked, but still had a high water content. The burlap remained on the slabs for 6 hours. The second batch, 041708, was cured similarly, except it had one variation. After the wet-burlaps were placed, they were monitored until they were completely dry and they were re-soaked to provide extra moisture to the surface of the slabs. A summary showing all of the best results for each batch is displayed in Table 7.5.2.2 and are discussed following the table.

Table 7.5.2.1 Mix Design for Batches Cured with Wet-Burlap

Batch Weights (yd^3)		
	Batch ID	
Material	040308	041708
Cement (lbs)	585	585
Rock (lbs)	1671	1671
Sand (lbs)	1349	1321
Water (lbs)	292	319
AEA (fl oz./cwt)	0	0
Retarder (fl oz./cwt)	0	0

Table 7.5.2.2 Summary Results using Wet-Burlap

Batch ID	Date Batched	Ambient Conditions		Compressive Strength (psi) (28 days)
		Max Temperature (°F)	Min Relative Humidity (%)	
040308	04/03/08	100	40	3825
041708	04/17/08	103	54	3500

7.5.2.1 **Curing Procedure and Total Crack Results.** The two batches were cured using wet burlap. The burlaps for each batch were submerged in water until saturated. Batch 041708 slabs were re-wet after the burlap had become dry. As with the previous tests, Slab A served as the control slab and was not cured. Unlike the other slabs, Slabs B, C, and D received identical curing. The average total area of cracks for each set of slabs is shown in Table 7.5.2.3 and photographs of the slabs are shown in Figures 7.5.2.1A and 7.5.2.1B.

Table 7.5.2.3 Area of Cracks per Slab

Batch ID	Area of Cracks (in^2)			
	SLAB A	SLAB B	SLAB C	SLAB D
Batch 040308	0.00	0.12	0.69	0.66
Batch 041708	0.12	0.36	0.79	0.60

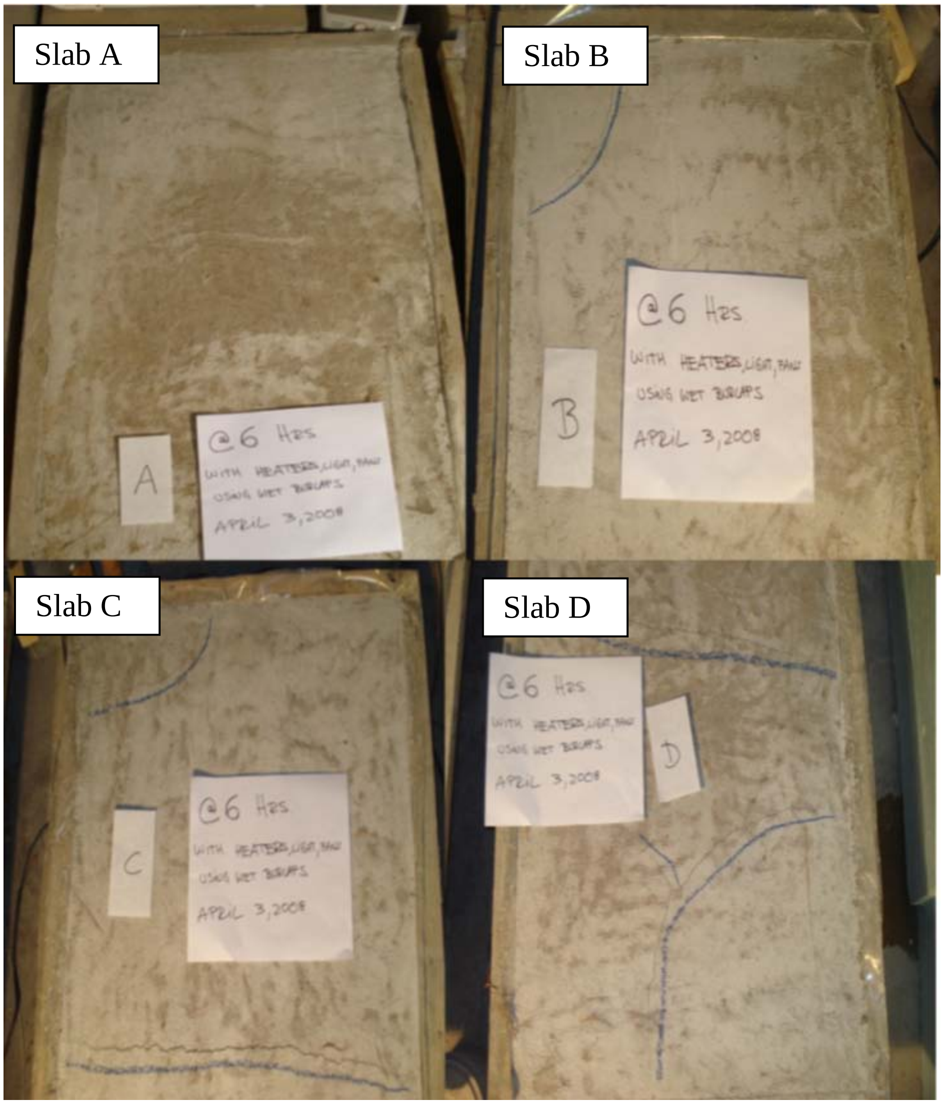

Figure 7.5.2.1A Slabs A through D for Batch No. 040308

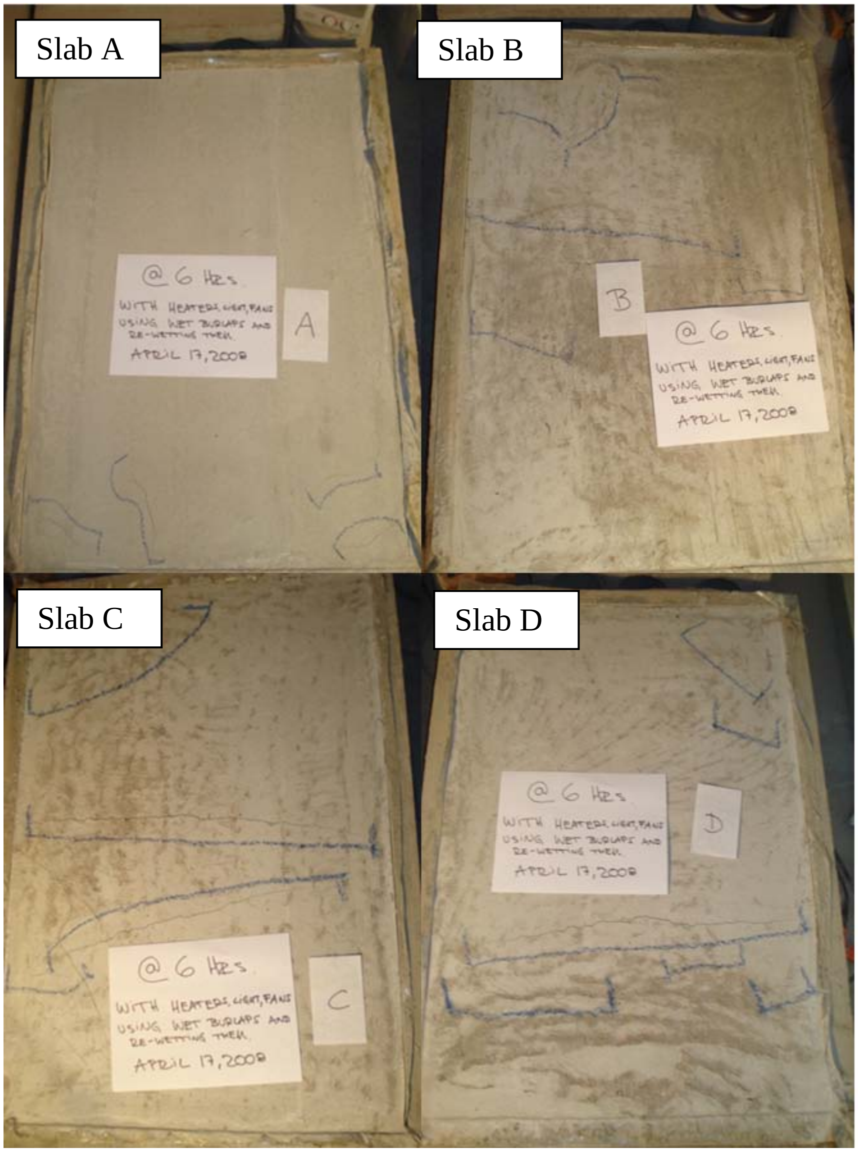

Figure 7.5.2.1B Slabs A through D for Batch No. 041708

Like the SINAK results, there were also many inconsistencies in the results from with the wet-burlap slabs. The highest area of cracks was 0.786 $in.^2$ for Slab C of Batch 041708 and Slab A of Batch 040308 did not crack. Although Slab A had no wet-burlaps applied to it, it did not show any cracks in its surface. This was a very irregular characteristic in this research. Zero cracks had been obtained previously with the SINAK curing compound, but this slab also had the highest dosage of curing compound. For this series, Slab A had no curing applied.

The factor that could have had the greatest effect was room temperature. For Batches 041708 and 040308 the slabs that cracked the least were A and B, and the slabs that cracked the most were C and D. Perhaps the testing chamber allowed for a temperature variation among the slabs. Also, the high traffic of people in the laboratory at the time of testing impeded the room to maintain a constant temperature throughout the test.

However, if we analyze the results for the other slabs we can see that they did not present a high area of cracks. The average area of cracks for the slabs that were cured in Batch 040308 was 0.490 $in.^2$ and for Batch 041708 was 0.581 $in.^2$. The difference between the maximum and minimum area of cracks in each batch was 0.688 $in.^2$ and 0.655 $in.^2$ for Batch 040308 and Batch 041708, respectively. The results for these two batches were very similar to each other. Batch 040308 had a standard deviation of 0.356 and Batch 041708 had a standard deviation of 0.288.

There is a bell-shaped trend to all four slabs in each batch. In each case, Slab A presents the minimum area of cracks followed by an increase in cracks in Slab B and Slab C. Then, Slab D shows a decrease in cracks, forming the bell-shaped curve. This bell-shaped trend is believed to relate only to this research investigation and is not a general trend for all or any concrete cured with wet-burlaps. Although the results for this curing process were different from one another, they provide a strong reason to believe that curing with wet-burlaps provides stable and almost uniform results when applied properly. Figure 7.5.2.1C shows the amount of cracks for each slab in both batches, displaying the bell-shaped curve discussed above.

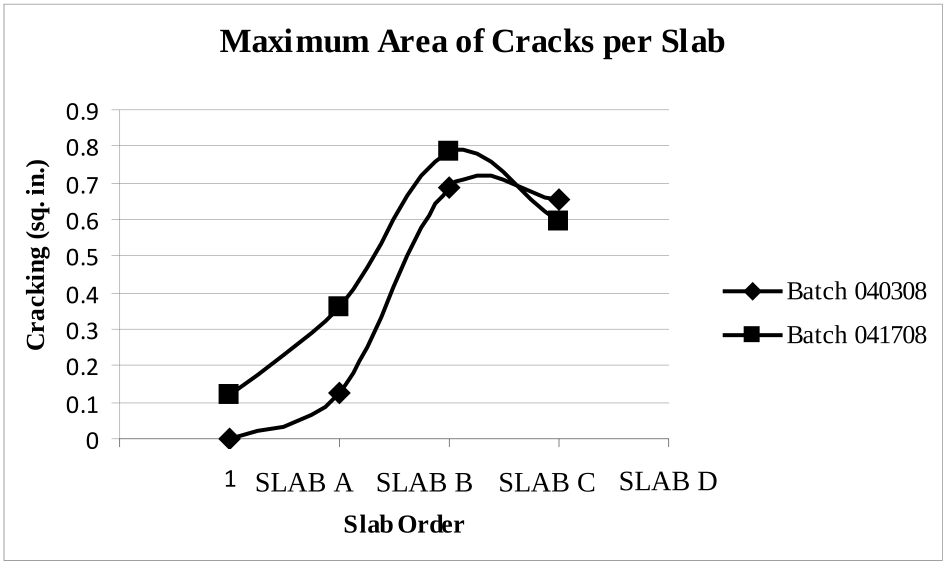

Figure 7.5.2.1C Trend of Maximum Area of Cracks per Slab

7.6 Conclusions

Due to the inconsistencies, few conclusions can be made. It appears that SINAK has the potential to reduce bridge deck cracking. Also, when curing with SINAK it is apparent that applying the manufacturer's recommended highest dosage achieves the best results. The slab with the highest curing compound dosage, Slab B, showed the least area of cracks in three out of four batches.

The researcher believes SINAK has a great potential in reducing plastic shrinkage cracks. The testing that was done throughout this research showed that SINAK does in fact reduce plastic shrinkage, and in some cases even prevents it. Developing a testing program that has stricter ambient temperature control is recommended since a big factor that affected this testing was the poor control of temperature in the laboratory. Also, a testing program that has a larger number of samples will be efficient to compare more results. Achieving this testing program will also assist in further testing with wet-burlap since the results in this investigation were not consistent and did not present the expected outcome.

CHAPTER 8

SUMMARY, CONCLUSIONS & RECOMENDATIONS

8.1 Summary of Research

Concrete bridge decks are highly susceptible to plastic shrinkage cracks due to their large exposed surfaces. When water evaporates from that surface faster or in a larger quantity than water bleeds to the surface from below, plastic shrinkage occurs, causing cracks if the shrinkage forces are larger than the tensile capacity of the green concrete. Shrinkage cracks allow water and salts to corrode the reinforcing steel and damage the concrete. Shrinkage cracking still occurs on new AHTD bridge decks, despite specifications that meet most available guidelines. The purpose of this research was to identify changes that AHTD could make to their specifications and procedures to reduce the incidence of shrinkage cracking in newly constructed concrete bridge decks.

The first task in the research project was a review of relevant literature on the topic. As previously mentioned, plastic shrinkage cracking has been and continues to be the topic of much research. Recent findings point to the possibility of constructing nearly crack free bridge decks (Darwin et al., 2005) by removing delays to curing such as tined finishes and curing compound. Others found that increased training and coordination among engineers and contractors produced the desired results. Yet, as more research is done, new and different problems may arise, so continued review of new findings is required.

The second task of this research was a survey to document the methods and materials currently used to construct bridge decks, mainly within Arkansas, but also in adjacent states. Of the 28 AHTD REs who responded to the survey, 12 believe that earlage cracking is a problem on Arkansas bridge decks and 12 do not. The other four respondents did not clearly answer the question. The anticipated patterns as to where and why bridge decks crack were not clear. The current methods and materials used were not consistent. The REs were concerned about the increased use of continuous placements that require retardants and about things such as tining the surface that cause delays to curing applications. Removing such impediments to curing and ambiguous specifications should produce more consistent results. The two adjacent state DOTs that responded held similar concerns and proposed similar solutions.

The third task of this project was to be evaluation of existing AHTD bridge decks to determine why some experienced early-age deck cracking and some did not. The surveys of ten bridge decks were completed using the DHDV to photograph the decks. Neither the automated processing of the images, nor subsequent manual processing, was able to measure cracking to the level of precision or accuracy required, nor was there sufficient historical information to accurately determine the causes of cracking recorded. This task was not fully addressed, but the methods used provide a basis for further development of this procedure in future research.

The fourth task was the observation and documentation of the concrete placement of five bridge decks in Arkansas. Monitoring included documenting everything from ambient conditions, concrete properties, construction procedures and material properties, curing and a post-cure examination of the new decks for cracking. Various interpretations of the specifications were observed. No single variable evaluated during this task consistently indicated the likelihood of cracking in the decks. Some variables did show some relationship to increased cracking; delayed curing, higher evaporation rate, and increased girder deflection.

The fifth and final task was a laboratory study in which various curing methods were applied to small scale slabs comprised of the paste portion of concrete mixtures similar to that used on AHTD bridge decks. Variations in curing procedures were evaluated based on the cracking measured on the slabs. As expected, the slabs that received wet cure or curing compound had less cracking than air-cured samples. However, comparisons between wet curing and membrane curing or the effects of curing compound application rate were not as clear. Additional testing is recommended.

8.2 Overall Conclusions

As previously noted, of the 28 REs responding to the survey (Task 2, Ch. 3), 43 percent reported that they believe bridge deck cracking is a problem in Arkansas and 43 percent reported that they do not. Of the five bridges observed in the field study (Task 4, Ch. 5), two bridges, 1 and 3, exhibited enough cracking that AHTD required repairs be made by the contractor, although bridge 3 repairs were partially driven by cracks in the unit not monitored under this research project.

Bridge deck 1 had the most cracks, 80; the widest average crack width, 0.013 inches; and the high density, 0.393 linear feet per square foot of measured deck. Bridge deck 1 also had the shallowest depth to span ratio, the second longest spans, the largest set-retarder dosage, and the longest time to final cure application. Bridge deck 4, which had the least cracking, ranks in the middle or lower in all these categories. All of these items (depth to span ratio, span length, set-retarder dosage and time to cure application) are indicated in the literature as factors contributing to plastic shrinkage and other forms of cracking in bridge decks.

Delayed curing is the predominant cause cited for shrinkage cracking in bridge decks in the research literature reviewed. This was also a source of concern for AHTD REs. Of the two decks with the most cracking observed in the field, one, Bridge 1, had the shortest time to curing compound application, but the longest time to final wet cure application. The other, Bridge 3, had the longest time to curing compound application but the shortest time to final cure. In the slab study results (Task 5, Ch. 6), the timely application of curing compound improved resistance to cracking and waiting too long allowed large cracks to form, regardless of the subsequent curing applied.

The tined surface finishes are a common source of these delays. Most researchers and some of the AHTD REs question whether waiting for the concrete to achieve proper tining consistency exposes the deck to too much risk of plastic shrinkage cracking. Other states are beginning to eschew tined finishes for a post-cure mechanically-grooved surface texture, to avoid delays to curing. During the casting of bridge deck 1, there were no long waits for tining to be completed prior to spraying the curing compound. On bridge deck 2, there was an extended period between the finishing operation and the tining to get the proper tined finish and again between the tining and the application of curing compound to avoid marring the tines. Bridge deck 2, had the second lowest cracking density, but other factors, such as low evaporation rate, may have mitigated the effects of the delays.

The amount of curing compound required to achieve the target application rate in the Task 5 (Ch 6) slab studies appeared to be much more than that applied to the decks in the field study (Task 4 Ch. 5). No standard test or research could be found for the field measurement of the application rate of curing compound and no attempt was made to measure the rate applied to any of the five decks in the field study. However, the application of compound to bridge deck 3 did appear to be particularly substandard. The use of a manual sprayer from one side of the 40 foot wide deck did not appear to provide the required coverage. Being that bridge deck 3 was the largest of the five, at approximately 15800 square feet, it is of particular interest. To achieve the AHTD specified rate of 125 ft^2/gal., this deck, excluding the area to be covered by the rail, would have required approximately 120 gallons of curing compound. Based on the photos from the placement, the sprayer used would hold approximately five gallons, requiring it to be refilled 24 times to complete the job. This was not likely the case. The other decks used sprayers connected to barrels of compound, making the application of an adequate quantity easier.

8.3 Recommendations

Inconsistent understanding and interpretations of AHTD construction plans and specifications regarding bridge deck placement have generated inconsistent results. AHTD should fully adopt the pre-placement meetings on all bridge decks with volumes exceeding 100 cubic yards. Based on the field study, experienced contractors can successfully place smaller decks without much input from the RE. However, new contractors and those making larger continuous pours should be required to formally discuss their plans for the placement, concrete and other materials to be used, equipment and personnel to be employed, and curing plans with the RE and project inspectors. If necessary, contractor and AHTD personnel involved could view a video, similar to the one MoDOT produced, detailing the desired outcome and emphasizing key criteria for a successful placement. Standardization of AHTD inspection and reporting procedures for decks to include more measurements, such as evaporation rate data and curing application, would also produce more consistent results in the field and a better source of information if problems arise.

AHTD should explore placing a limit on the size of bridge that can be continuously placed. Bridge 1, which was a 331 yd^3 pour and continuously cast, had locations in the deck that had compressive strengths of 130 psi and 2590 psi at 2 days of age. This variation in strength affects bridge stiffness which can lead to early age cracking. By first casting the positive moment regions of the bridge deck followed by the negative moment regions, it would decrease placement time which would allow for the curing to be applied earlier and eliminate the need for or reduce the set retarder dosage rate which would also allow the curing regimen to be applied earlier. Differences in concrete strength would also be minimized if set retarder dosages were decreased.

Tined surface treatments should be replaced with post-cure mechanically grooved texture, at least in an experimental selection of decks, to evaluate the cost and performance. This would allow for quicker application of the final cure. Literature findings also indicate that curing compound applied after the seven day wet cure helps to reduce drying shrinkage.

Whether used before or after the wet cure, curing compound must be applied at the proper rate to be effective. Actual field measurement of the coverage would be difficult to achieve, but calculating the amount required and measuring the amount used at the source would provide a good estimate of the coverage applied. Additionally, prescriptive equipment specifications could be made based on manufacturers' recommendations, to ensure even coverage. The curing compound SINAK showed promise in reducing cracking and additional research should be conducted to further examine the performance of SINAK.

Additional research is required to standardize a test of curing systems that encompasses enough of the variables to be a realistic simulation, but controls enough of the variables to show clear results. The test must have mixture, proportional, and environmental similarities to the real thing. With such a test, additional research could show the effectiveness of different curing sequences, materials and timings at reducing plastic shrinkage cracking in concrete bridge decks.

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APPENDIX A

A.1 Task 2 Survey Responses

Due to space and printing considerations, the complete responses to the Task 2 survey of AHTD Resident Engineers and Surrounding State DOTs are not included in this printing of this thesis. The complete text of these responses is available in electronic or printed form from either of the following sources:

Micah Hale, PE, Ph D
Department of Civil Engineering
4190 Bell Engineering Center
University of Arkansas
Fayetteville, AR, 72701
Email: micah@uark.edu
Phone: (479) 575-6348

Steven Peyton, PE
Bridge Division
Arkansas Highway and Transportation Department
P.O. Box 2261
Little Rock, AR 72203-2261
Email: Steven.Peyton@arkansashighways.com
Phone: (501)569-2000

A.2 AHTD District 4 Bridge Deck Placement Inspection Form

Figure A.2.1 AHTD District 4 Bridge Deck Placement Inspection Form

Job #: ________ Date: ________ Bridge #: ________ Span #/pour sequence # ________

Plan quantity for pour: ________ Station ________ to ________ Screed type: Longitudinal/Transverse

Actual quantity for pour: ________ Station ________ to ________ Screed brand: ________

Number of concrete loads: ________ Station ________ to ________ Production(CY/Hr.): ________

Weather: Sunny/Ptly cloudy/Cldy/Drizzle/Rain Temperature: Begin: ________ End: ________

Wind: Calm/Gusty/breezy/Strong Time of day: Begin: End:

QUALITY CONTROL

Cast by: Number of cylinders:

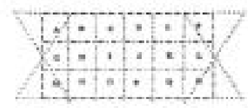
Time cast: % air: Slump: ________

Concrete load #: Truck #:

ADMIXTURE (Per 100 lbs. of cement):

Oz. retardant : Oz. air entraining:

Air temperature: Concrete temperature:



Location of load in deck

Cast by: ________ Number of cylinders: ________

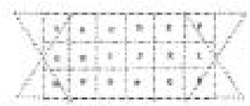
Time cast: ________ % air: ________ Slump: ________

Concrete load #: ________ Truck #: ________

ADMIXTURE (Per 100 lbs. of cement):

Oz. retardant : ________ Oz. air entraining: ________

Air temperature: ________ Concrete temperature: ________



Location of load in deck

Cast by: ________ Number of cylinders: ________

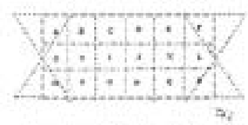
Time cast: ________ % air: ________ Slump: ________

Concrete load #: ________ Truck #: ________

ADMIXTURE (Per 100 lbs. of cement):

Oz. retardant : ________ Oz. air entraining: ________

Air temperature: Concrete temperature:



Location of load in deck

IAS

Cast by: ________ Number of cylinders: ________

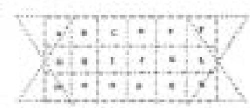
Time cast: ________ % air: ________ Slump: ________

Concrete load #: ________ Truck #: ________

ADMIXTURE (Per 100 lbs. of cement):

Oz. retardant : ________ Oz. air entraining: ________

Air temperature: Concrete temperature:



Location of load in deck

COMMENTS

Placement: Crane/Pump

Vibration: ________

Screed: ________

Finishing: ________

Tining: ________

Curing: Membrane/Mats

Temperature of deck during 5 day curing:					Notes:
Day 1	________	a.m.	________	p.m.	
Day 3	________	a.m.	________	p.m.	
Day 2	________	a.m.	________	p.m.	
Day 4	________	a.m.	________	p.m.	________
Day 5	________	a.m.	________	p.m.	

NOTE: This form is to be completed for each deck pour and submitted at the end of 5 day curing period.

Completed by: ________

APPENDIX B

Task 4 Field Study Data

B.1 Bridge 1: Mill Creek Road over I-40 (AHTD Job R80072)

Table B.1.1 Condition Records from Deck Placement

Job	R80072		Mill Creek Road Overpass		
Bridge			**Span/Unit**		
Date	June 15, 2005		**Start Time**		5:45:00 AM
Time	**Air Temp (°F)**	**Wind (mph)**	**RH (%)**	**WB Temp (°F)**	**Notes**
5:45 AM	68	0	51		Trucks on Job
6:00 AM	69	0	51.9	58.8	
7:00 AM	85.9	0	71.6	61.3	After 1st FP
7:30 AM	83.8	0	69.4	75.9	
8:00 AM	86.5	0	67.8	75.5	Before 2 FP
10:00 AM	91.7	0	47.1	75.9	1/2 Way
10:08 AM	93.9	0	46.1	77.1	Extra
11:00 AM	93.8	2.5	52.6	73.9	Almost 3rd Sample
2:10 PM	95	4	52.3	80.4	No Curing Yet

Table B.1.2 Cracking Records from Deck Mapping

AHTD Project #	**R80072**		**Project Name**	**Mill Creek Rd**
Bridge #	**7007**	**Pour #**	**NA**	
Deck #	**1**	**Pour Date**	**June 15,2005**	
Survey Date	**4/5/2006**	**Time**	**10:00 AM**	
Location	**SB Lane**	**Weather**	**Clear**	
Length (ft)	**50**	**Area (ft^2)**	**600**	
Width (ft)	**12**			

Crack ID Number	**Type T, L, D, M**	**Max Width (in.)**	**Length (ft.)**	**Comments**
1	D	0.012	0.5	
2	L	0.01	1	
3	L	0.012	0.5	
4	L	0.01	1	
5	L	0.01	1	
6	D	0.01	0.5	
7	L	0.01	2	W/ TRANSVERSE CURVING
8	L AND T	0.01	1	TRANSVERSE & TURNS LONG, SEALED
9	L	0.01	1	ALSO, 2 OR 3 PARALLEL CRACKS
10	L	0.01	1	
11	T	0.007	1	
12	L	0.01	1	
13	L	0.02	8	SEALED
14	L	0.024	7	SEALED
15	L	0.012	33	SEALED
16	L	0.016	36	
17	L	0.007	17	
18	L	0.007	1.25	
19	L	0.012	2	
20	L	0.005	2	SEALED
21	L AND T	0.012	6	SEALED
22	L	0.01	2	SEALED
23	L	0.01	2.5	
24	L	0.007	0.33	
25	L	0.02	2	
26	L	0.012	48	SEALED, GROUPED
26	T	0.012	15	
27	M	0.013621302	42.25	
	Total Length (ft)		235.83	
	Crack Density(ft/ft)		0.39305	
	Avg. Width (in)		0.013003265	

B.1 Bridge 2: I-40 Over Hwy 365, 176, UPRR (AHTD Job B60117)

Table B.2.1 Condition Records from Deck Placement

Job	B60117		Levy Overpass		
Bridge	B6909		**Span/Unit**	1&2/1	
Date	7/20/2005		**Start Time**		8:40PM
Time	**Air Temp (°F)**	**Wind (mph)**	**RH (%)**	**WB Temp (°F)**	**Notes**
9:05 PM	89.4	1.5	64	79.7	
9:30 PM	88.3	0	68.1	79.7	
10:05 PM	86.9	1	69.7	78.6	
11:07 PM	84.5	1	68.2	76.2	
11:50 PM	85.4	0	76.5	78.9	
12:25 AM	83.6	0	77	77.7	
12:55 AM	84	0	75	77.5	
2:45 AM	83.4	0	72.6	75.9	
3:00 AM	84.2	0	77.4	78.2	

Table B.2.2 Cracking Records from Deck Mapping

AHTD Project #	B60117		**Project Name**	**Levy Overpass**
Bridge #	**B6909**	**Pour #**	**Phase 2 1**	
Deck #	**2**	**Pour Date**	**7/20/2005**	
Survey Date	**8/1/2005**	**Time**	**11:00 AM**	
Location	**EB Lane**	**Weather**	**Hot/Clear**	
Length (ft)	**330**	**Area (ft^2)**	**10367**	
Width (ft)	**33.4**			

Crack ID Number	**Type T, L, D, M**	**Max Width (in.)**	**Length (ft.)**	**Comments**
1	T	0.005	9	
2	T	0.002	17	
3	T	0.002	9	
4	T	0.002	9	
5	T	0.002	9.5	
6	T	0.005	8	
7	T	0.002	9.5	
8	L	0.002	1	In Gutter
9	T	0.002	27	
10	L	0.002	5	In Gutter
11	T	0.002	4	
12	L	0.002	5	
13	L	0.002	9	
14	L	0.002	3	In Gutter
15	T	0.016	0.25	May be Plastic Shrinkage
16	L	0.002	3	In Gutter
	Total Length (ft)		128.25	
	Crack Density(ft/ft)		0.012370375	
	Avg. Width (in)		0.00242495	

B.3 Bridge 3: Ouachita River (AHTD Job 060938)

Table B.3.1 Condition Records from Deck Placement

Job	060938		Ouachita River		
Bridge	04875		**Span/Unit**	1	
Date	Aug. 24, 2005		**Start Time**		3:15AM
Time	**Air Temp (°F)**	**Wind (mph)**	**RH (%)**	**WB Temp (°F)**	**Notes**
5:05 AM	75.9	0	73	79.7	
6:08 AM	77.7	0	71.3	79.7	
7:05 AM	80	0	68.2	78.6	
8:35 AM	86.9	0	74.5	76.2	
9:30 AM	85	0	73	78.9	
10:30 AM	90	0	74.2	77.7	
11:30 AM	95.5	0	56.9	77.5	
12:00 PM	92	0	59.5	75.9	
5:05 AM	75.9	0	73	79.7	

Table B.3.2 Cracking Records from Deck Mapping

AHTD Project #	060938		**Project Name**	Ouachita River
Bridge #	04875	**Pour #**	1	
Deck #	3	**Pour Date**	Aug. 24,,2005	
Survey Date	1/27/2006	**Time**	10:00 AM	
Location	WB Lane	**Weather**	Cloudy 43°F	
Length (ft)	100	**Area (ft^2)**	1200	
Width (ft)	12			

Crack ID Number	**Type T, L, D, M**	**Max Width (in.)**	**Length (ft.)**	**Comments**
1	M			
2	T	0.007	9	
3	L	0.002	9	
4	T	0.005	6	
5	T	0.007	4	
6	L	0.002	6	
7	T	0.01	8	
8	T	0.007	9	
9	T	0.007	7	CONTINUES TO PARAPET
10	T	0.005	7	
11	T	0.005	8	
12	T	0.007	10	
13	T	0.007	12	FULL WIDTH OF DECK
14	T	0.005	4	LARGER IN OVERHANG
15	T	0.007	9	
16	T	0.005	4	CONTINUES TO GUTTER LINE
17	T	0.007	8	
18	T	0.005	3	GOES TO RAIL
19	T	0.007	9	
20	T	0.005	3	CONTINUES TO GUTTER LINE
	Total Length (ft)		135	
	Crack Density(ft/ft)		0.1125	
	Avg. Width (in)		0.0061037	

B.4 Bridge 4: SH 15 over Main Ditch (AHTD Job 020384)

Table B.4.1 Condition Records from Deck Placement

Job	020384		SH 15 over Main Ditch		
Bridge	07024		**Span/Unit**		
Date	Sept. 7, 2005		**Start Time**		6:00 AM
Time	**Air Temp (°F)**	**Wind (mph)**	**RH (%)**	**WB Temp (°F)**	**Notes**
7:00 AM	67.2	0	70.5	59.9	
8:00 AM	72.8	0	66.4	66.5	
9:40 AM	84.2	0	63	74.8	
10:17 AM	94.1	0	43.7		
11:25 AM	91.4	0	48.8		
2:18 PM	94.1	0	38.8		
3:00 PM	94.2	0	40.8		

Table B.4.2 Cracking Records from Deck Mapping

AHTD Project #	020384		**Project Name**	Main Ditch
Bridge #	7024	**Pour #**	NA	
Deck #	4	**Pour Date**	Sept. 7,2005	
Survey Date	2/8/2006	**Time**	2:00 PM	
Location	SB Lane	**Weather**	Clear	
Length (ft)	100	**Area (ft^2)**	1200	
Width (ft)	12			

Crack ID Number	**Type T, L, D, M**	**Max Width (in.)**	**Length (ft.)**	**Comments**
1	L	0.002	0.42	
2	T	0.01	1.5	
3	D	0.005	0.5	
4	T	0.005	0.5	
5	T	0.01	4	
6	L	0.01	0.5	
	Total Length (ft)		7.42	
	Crack Density(ft/ft)		0.006183333	
	Avg. Width (in)		0.00887332	

B.5 Bridge 5: US 70 over Bevins Bayou(AHTD Job 110388 site 2)

Table B.5.1 Condition Records from Deck Placement

Job	110388s2		US 70 over Bevins Bayou		
Bridge	07016		**Span/Unit**		
Date	Sept. 23, 2005		**Start Time**		7:05 AM
Time	**Air Temp (°F)**	**Wind (mph)**	**RH (%)**	**WB Temp (°F)**	**Notes**
7:05 AM	71.7	0	58.2		
7:19 AM	73.4	0	67.1		
8:27 AM	83.4	1.5	64.1	74.3	
9:15 AM	88.5	1.75	57.1	76.8	
10:27 AM	90.6	1.5	57		
11:25 AM	95.7	2.5	44.7	79.8	
12:45 PM	94.8	6	43.9	77.9	
2:15 PM	96.4	6	42.6	78	

Table B.5.2 Cracking Records from Deck Mapping

AHTD Project #	110388 site 2		**Project Name**	Bevins Bayou
Bridge #	7016	**Pour #**	NA	
Deck #	5	**Pour Date**	Sept. 23,2005	
Survey Date	2/9/2006	**Time**	9:00 AM	
Location	EB Lane	**Weather**	Cloudy 40 °	Temp Failing/ Rain Starts
Length (ft)	65	**Area (ft^2)**	780	
Width (ft)	12			

Crack ID Number	**Type T, L, D, M**	**Max Width (in.)**	**Length (ft.)**	**Comments**
1	D	0.007	2	
2	L	0.005	1.5	
3	L	0.002	5	
4	T	0.002	7	
5	T	0.002	7	
6	T	0.002	4	
7	T	0.01	2	
8	D	0.01	7	
9	D	0.005	3	
10	L	0.01	1	
	Total Length (ft)		39.5	
	Crack Density(ft/ft)		0.050641026	
	Avg. Width (in)		0.00462025	

APPENDIX C

Task 5 Lab Study Data

C.1 Thin Slab Study Batch R-4 Data

Table C1.1 Batch R-4 Fresh Properties and Compressive Strength Data

Mix ID: **R-4**

DATE BATCHED	5/18/06			
TIME OF BATCH	11:30			
DESCRIPTION	Retarded 4 fl oz/cwt Thin Slabs			
	Control	Tines & Burlap	Tines & CC	T & CC & B
Slab ID	A	B	C	D
Time Placed	12:10 PM	12:30 PM	12:53 PM	12:45 PM
Time Screeded	12:15 PM	12:37 PM	12:57 PM	12:50 PM
Time Finished	12:25 PM	12:40 PM	12:59 PM	1:01 PM
Time Tined	X	12:45 PM	1:00 PM	1:02 PM
Time 1st CC	X	X	1:29 PM	1:29 PM
Time 2ndCC	X	X	2:35 PM	2:35 PM
Time Wet Burlap	X	3:09 PM	X	3:09 PM
Time of Wind Start	1:35 PM	1:35 PM	1:35 PM	1:35 PM
Time of First Crack	3:59 PM	3:10 PM	no cracking	no cracking

Flow Table		Cone Base	101mm
Measurement Time	Reading 1	Reading 2	Reading 3
12:03 PM	200	210	205
12:30 PM	198	191	192
1:10 PM	183	190	190

Compressive Strength	DAY 1	DAY 7	DAY 28
DATE	5/19/06	5/25/06	6/15/06
TIME	11:58 AM	11:52 AM	2:11 PM
	psi	psi	psi
	pounds	pounds	pounds
1	312	4307	5367
	3920	54120	67440
2	425	3971	5570
	5340	49900	69990
3	467	4120	5310
	5870	51770	66730

Table C1.2 Batch R-4 Curing Conditions Data

Job	**R-4**					
Date	**5/18/2006**		**Start Time**		**11:30 AM**	**Batch Time**
Date	**Time**	**Air Temp**	**Wind**	**RH**	**Conc. Temp**	**Notes**
5/18/2006	1:31 PM	75.4	10	51	68	Fans On
	1:37 PM	76.8	10	39	76.8	
	2:28 PM	78.6	10	33	78.6	Closed Curing Room
	3:18 PM	84.9	10	36	84.9	
	3:58 PM	89.1	10	31	89.1	
	4:30 PM	91.2	10	30	91.2	Rewet Burlap
	5:30 PM	92.1	10	30	92.1	Fans Off
5/19/2006	9:00 AM	74.1	10	55	74.1	On
	9:35 AM	84.9	10	42	84.9	
	11:06 AM	89.4	10	32	89.4	
	12:25 PM	91.4	10	31	91.4	Started Maps
	2:56 PM	96.6	10	31	96.6	
	3:40 PM	95.3	10	30	95.3	Off
5/20/2006	10:25 AM	77	10	57	77	on
	10:50 AM	89.2	10	45	89.2	
	6:25 PM	104.2	10	31	104.2	Off - Saturday
5/21/2006						Sunday
5/22/2006	11:00 AM	79.9	10	66	79.9	On No Change in Cracking
	11:55 AM	93.4	10	43	93.4	
	12:47 PM	95.9	10	40	95.9	
	4:20 PM	102.2	10	32	102.2	Re-wet Burlap
	5:33 PM	101.3	10	35	101.3	Off
5/23/2006	10:15 AM	80.6	10	63	80.6	On Re-wet Burlap
	11:00 AM	93.2	10	48	93.2	
	1:47 PM	100.4	10	33	100.4	
	3:47 PM	100.2	10	36	100.2	
	4:20 PM	102.9	10	34	102.9	Off Re-wet Burlap
5/24/2006	8:30 AM	80.4	10	63	80.4	On
	10:00 AM	93.7	10	47	93.7	
	11:30 AM	98.4	10	41	98.4	
	1:00 PM	101	10	38	101	Re wet Burlap
	2:00 PM	99.3	10	42	99.3	Off
5/25/2006	10:00 AM	80.4	10	61	80.4	On Re-wet Burlap
	10:30 AM	91.9	10	51	91.9	
	12:00 PM	98.8	10	41	98.8	
	1:50 PM	100.6	10	40	100.6	
	5:56 PM	105.4	10	37	105.4	Off

Figure C1.1 Crack Map for Slab R-4A

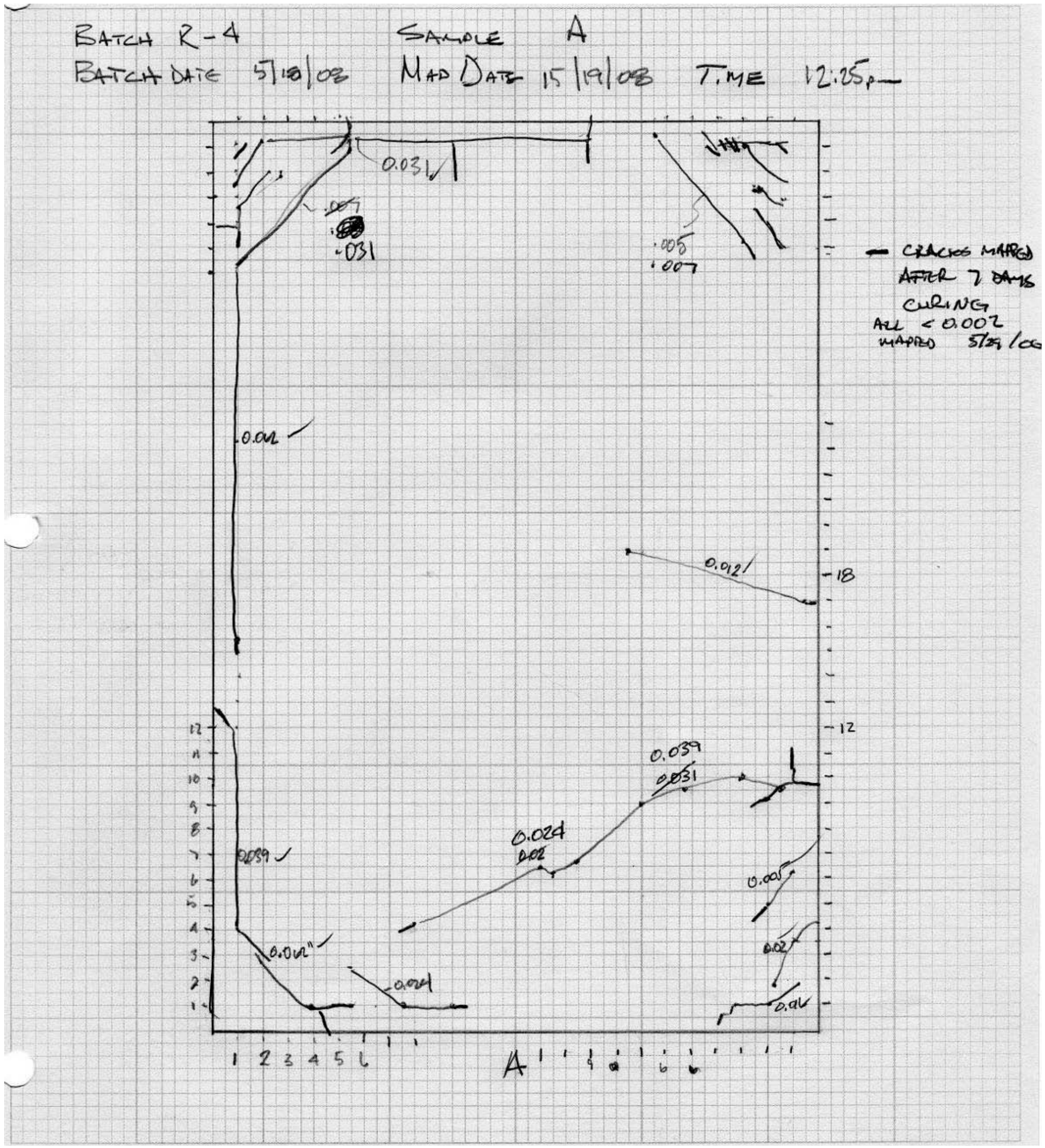


Figure C1.2 Crack Map for Slab R-4B

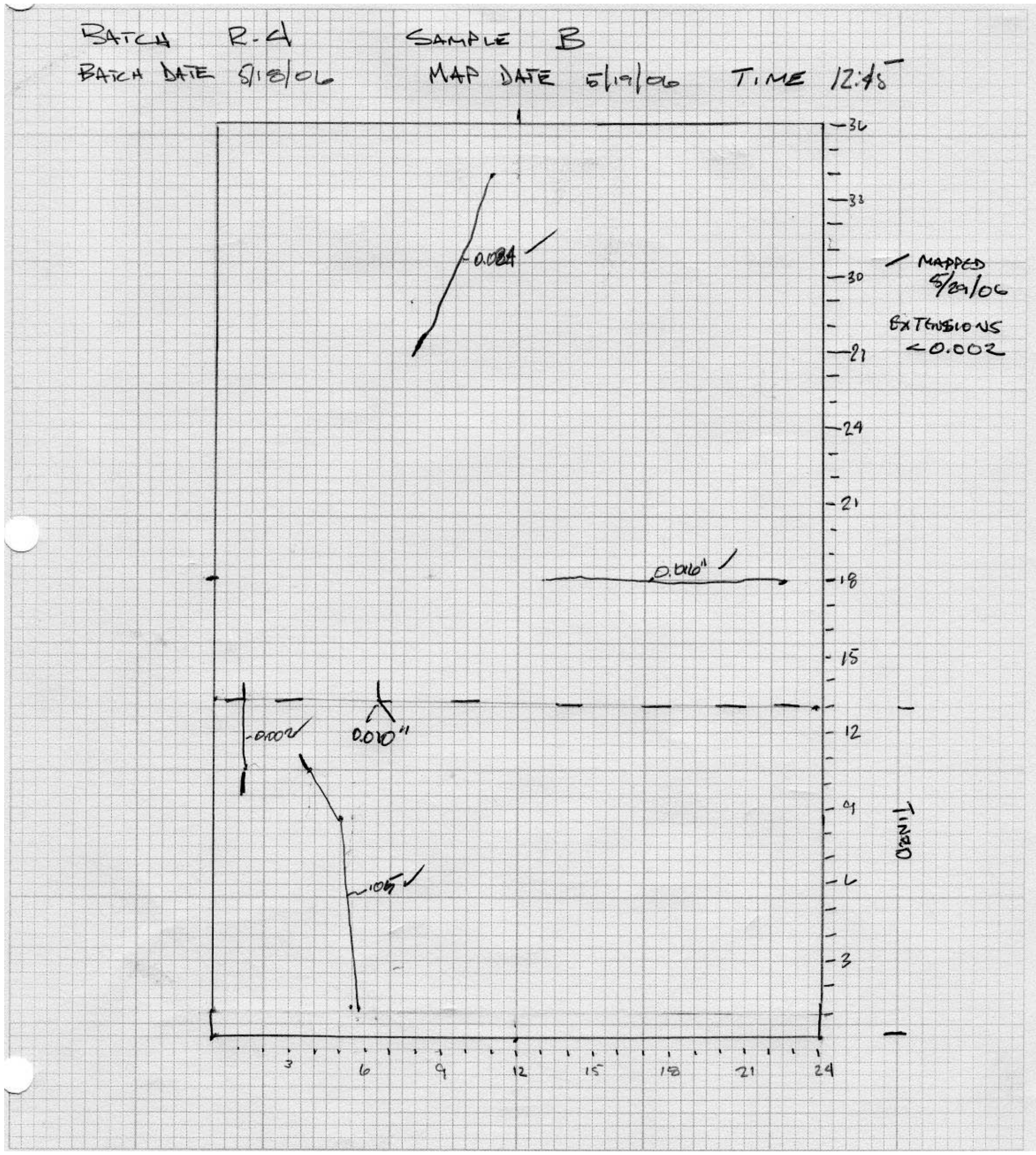


Figure C1.3 Crack Map for Slab R-4C

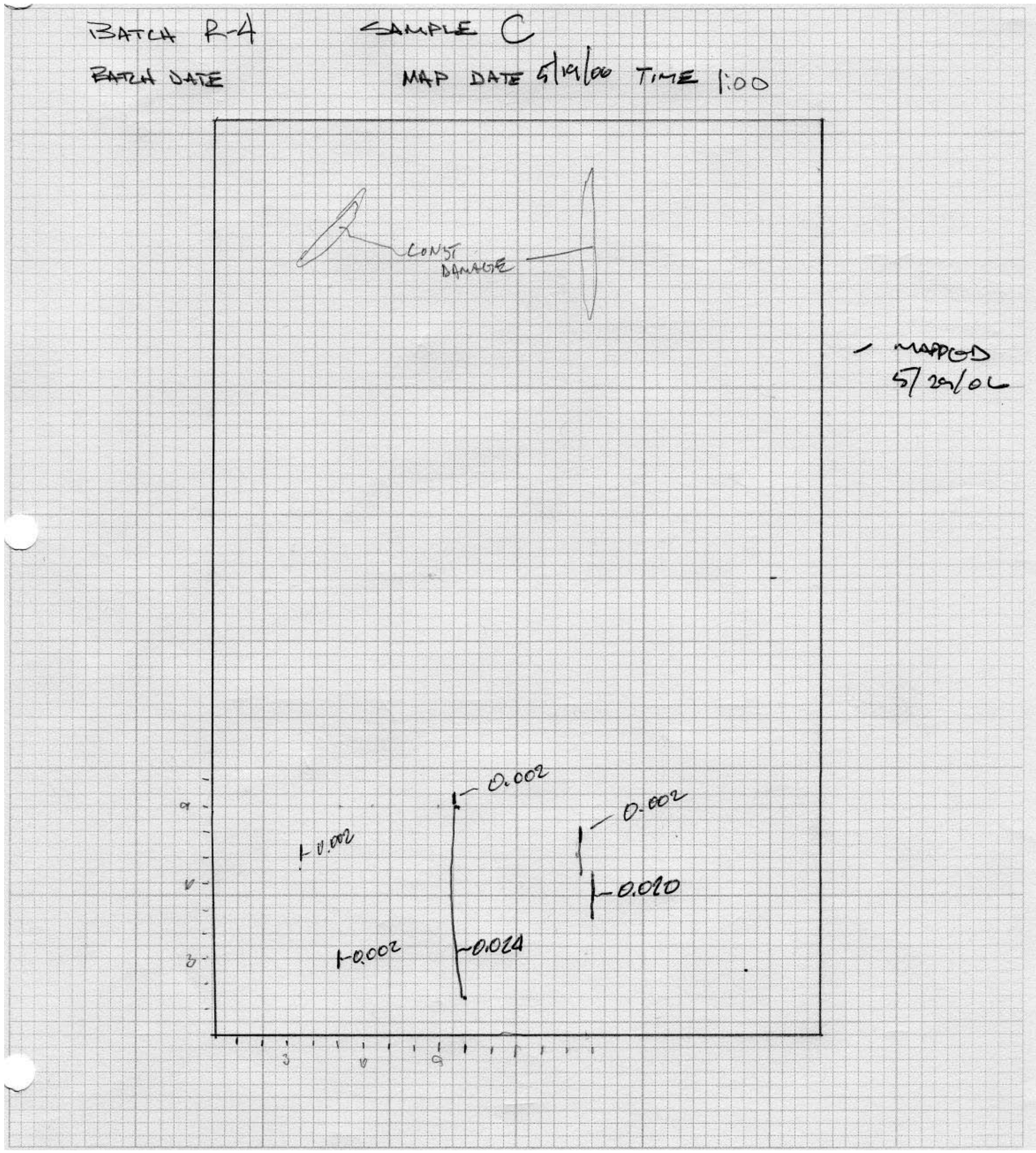
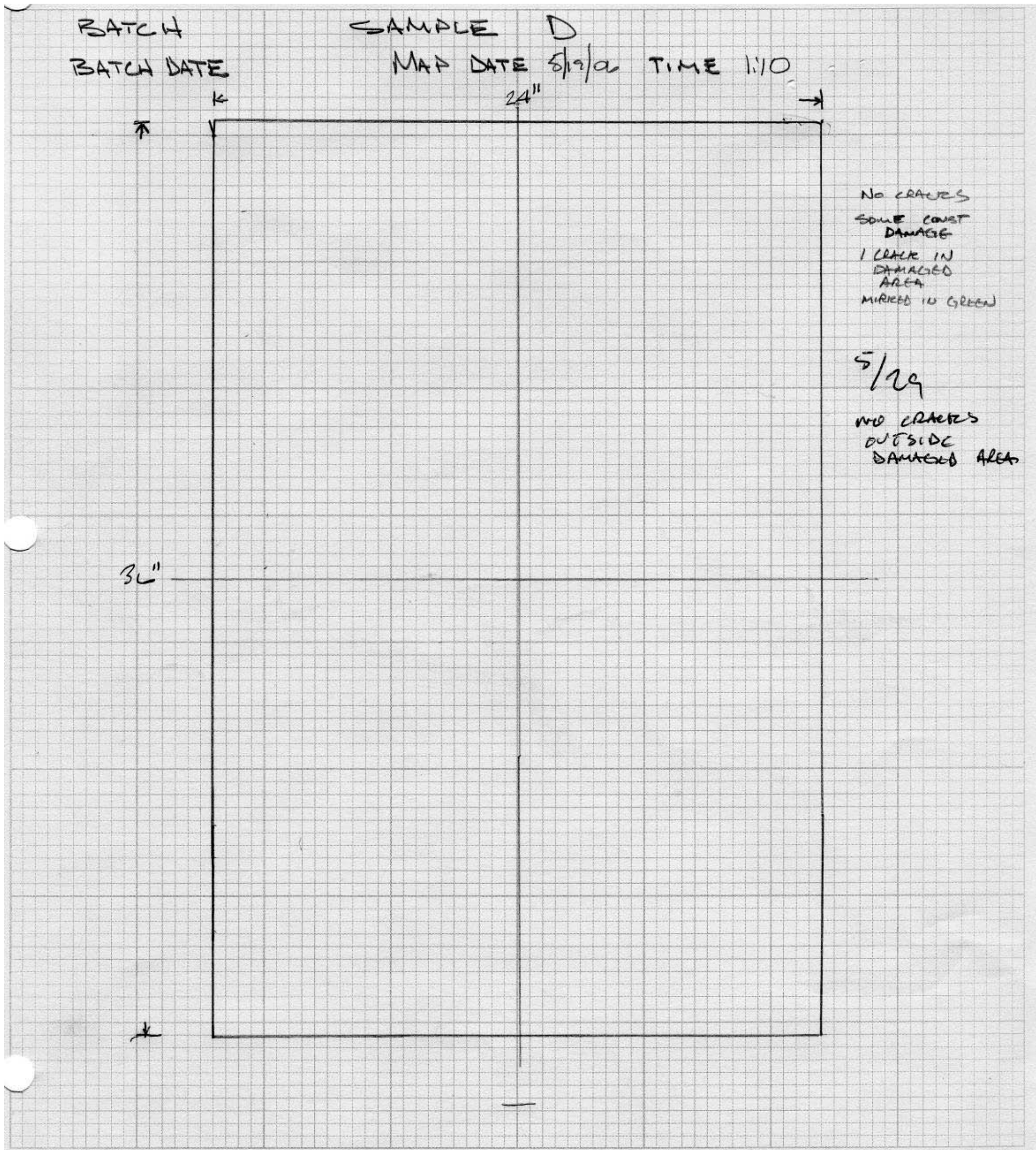


Figure C1.4 Crack Map for Slab R-4D



C.2 Thin Slab Study Batch R-8 Data

Table C2.1 Batch R-8 Fresh Properties and Compressive Strength Data

Mix ID: **R-8**

DATE BATCHED	5/30/06			
TIME OF BATCH	10:15			
DESCRIPTION	Retarded 8 fl oz/cwt Thin Slabs			
	Control	Tines & Burlap	Tines & CC	T & CC & B
Slab ID	A	B	C	D
Time Placed	11:15 AM	11:20 AM	11:33 AM	11:43 AM
Time Screeded	11:17 AM	11:29 AM	11:36 AM	11:45 AM
Time Finished	11:18 AM	11:34 AM	11:42 AM	11:50 AM
Time Tined	X	11:50 AM	11:51 AM	11:52 AM
Time 1st CC	X	X	12:14/12:44	12:16 / 12:45
Time 2ndCC	X	12:08 PM	X	12:10 PM
Time Wet Burlap	12:05 PM	12:05 PM	12:05 PM	12:05 PM
Time of Wind Start	1:53 PM	not recorded	not recorded	not recorded
Time of First Crack	11:15 AM	11:20 AM	11:33 AM	11:43 AM
Flow Table		Cone Base	101mm	
Measurement Time	Reading 1	Reading 2	Reading 3	Reading 4
11:05 AM	208	212	214	216
11:33 AM	192	198	195	197
12:00 PM	176	185	184	182

Compressive Strength	DAY 1	DAY 7	DAY 28
DATE	5/31/06	6/6/06	6/27/06
TIME	11;30	9:40	13:00
	psi	psi	psi
	pounds	pounds	pounds
1	59	79	4371
	730	992	54930
2	40	50	4793
	500	630	60230
3	41		4748
	520		59660

Table C2.2 Batch R-8 Curing Conditions Data

Job	**R-8**					
Date	**5/30/2006**		**Start Time**		**10:15 AM**	**Batch Time**
Date	**Time**	**Air Temp**	**Wind**	**RH**	**Conc. Temp**	**Notes**
5/30/2006	12:05 PM	78.6	10	72	72	Fans & Heat On 1st CC - C&D WB on B
	12:35 PM	89.4	10	57		2nd CC WB on D
	1:15 PM	91.6	10	51	85	
	2:54 PM	94.3	10	56		
	3:23 PM	95.9	10	50	90	
	4:04 PM	97.5	10	48		
	5:00 PM	100.2	10	42		
5/31/2006	7:40 AM	77.9	10	65	78	On Re-wet Burlap
	8:10 AM	89.4	10	54		
	9:40 AM	95.7	10	45		
	11:22 AM	99.3	10	41	98	
	12:05 PM	100.4	10	40	100	Heat Off- Mapped Cracks
	12:45 PM	85.8	10	57	90	Heat on Finished Mapping RWB
	2:20 PM	99.3	10	41		Off
6/1/2006	9:00 AM	77.7	10	72	78	On Re-wet Burlap
	10:00 AM	91.9	10	56	90	
	12:24 PM	99.3	10	44		
	12:55 PM	100.2	10	40		RWB
	1:57 PM	97.9	10	47		
	2:54 PM	99.3	10	44		
	3:30 PM	98.8	10	44	100	Off RE-wet Burlap
6/2/2006	10:00 AM	77.7	0	62	78	On RWB
	10:40 AM	87.6	10	53		
	11:55 AM	93	10	42		
	12:40 PM	95.5	10	35		
	2:00 PM	97.9	10	32		
	3:17 PM	100.2	10	31	100	
	4:05 PM	100.2	10	30	100	Off Map Cracks

Figure C2.1 Crack Map for Slab R-8A

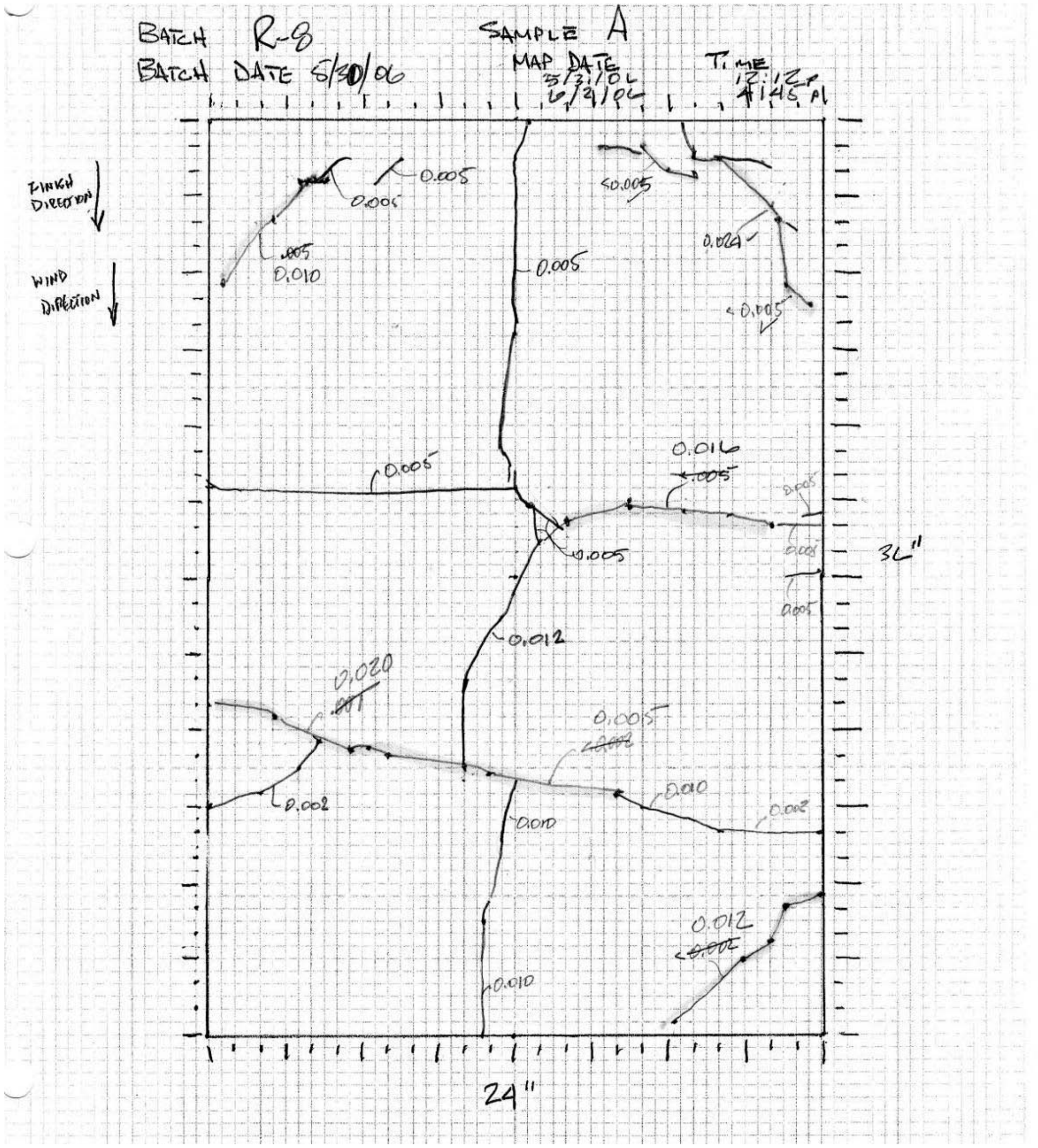


Figure C2.2 Crack Map for Slab R-4B

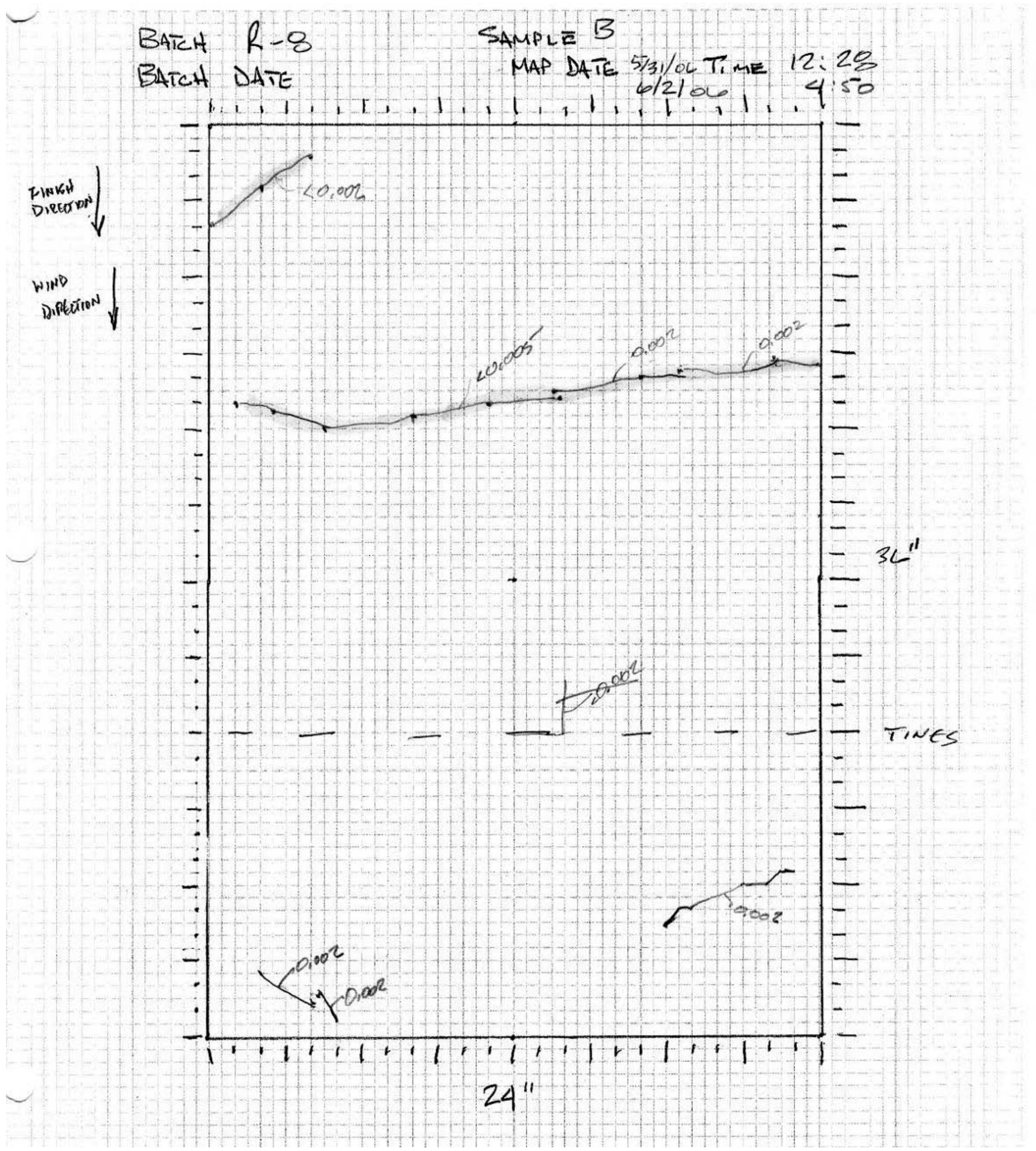


Figure C2.3 Crack Map for Slab R-8C

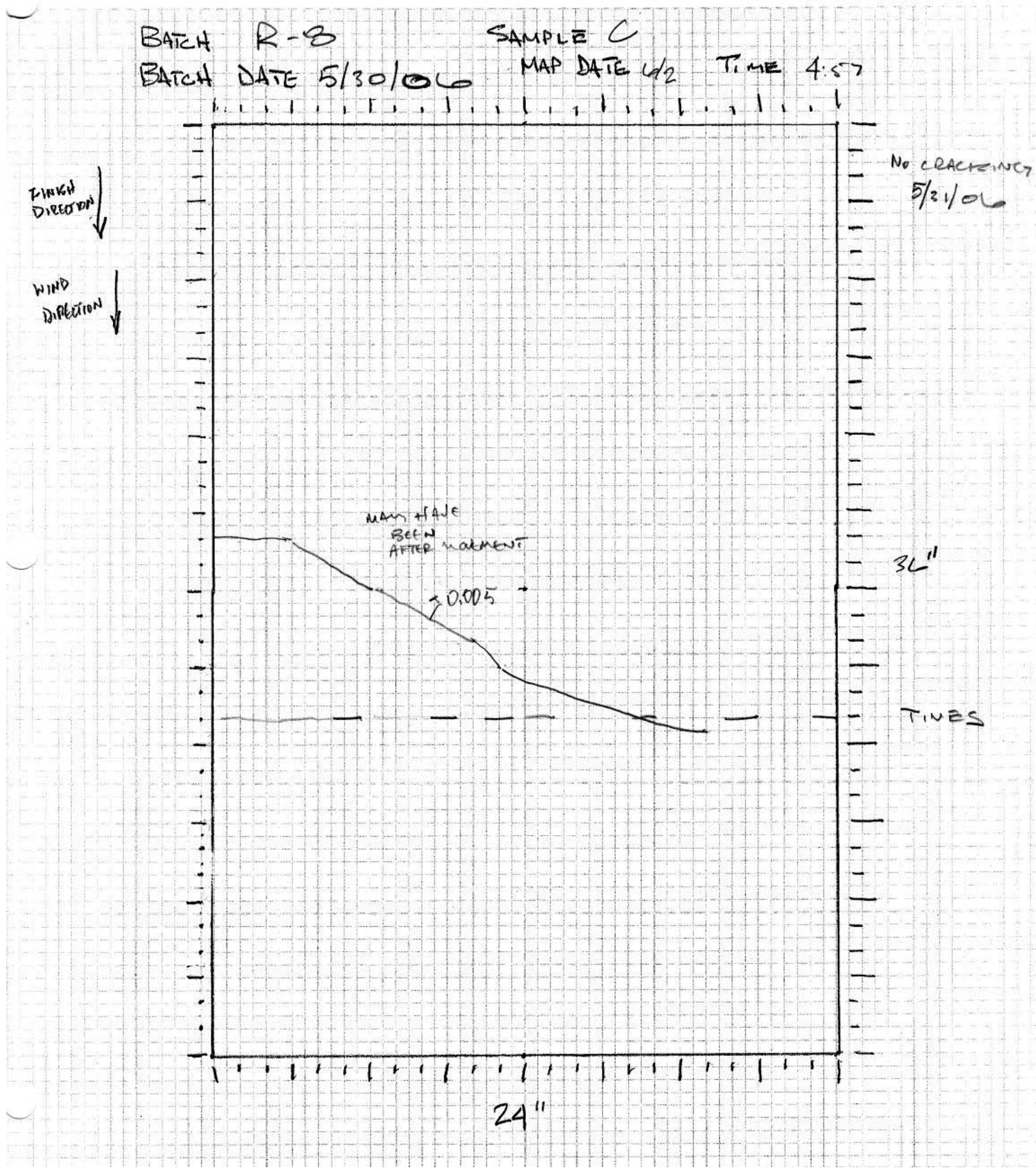
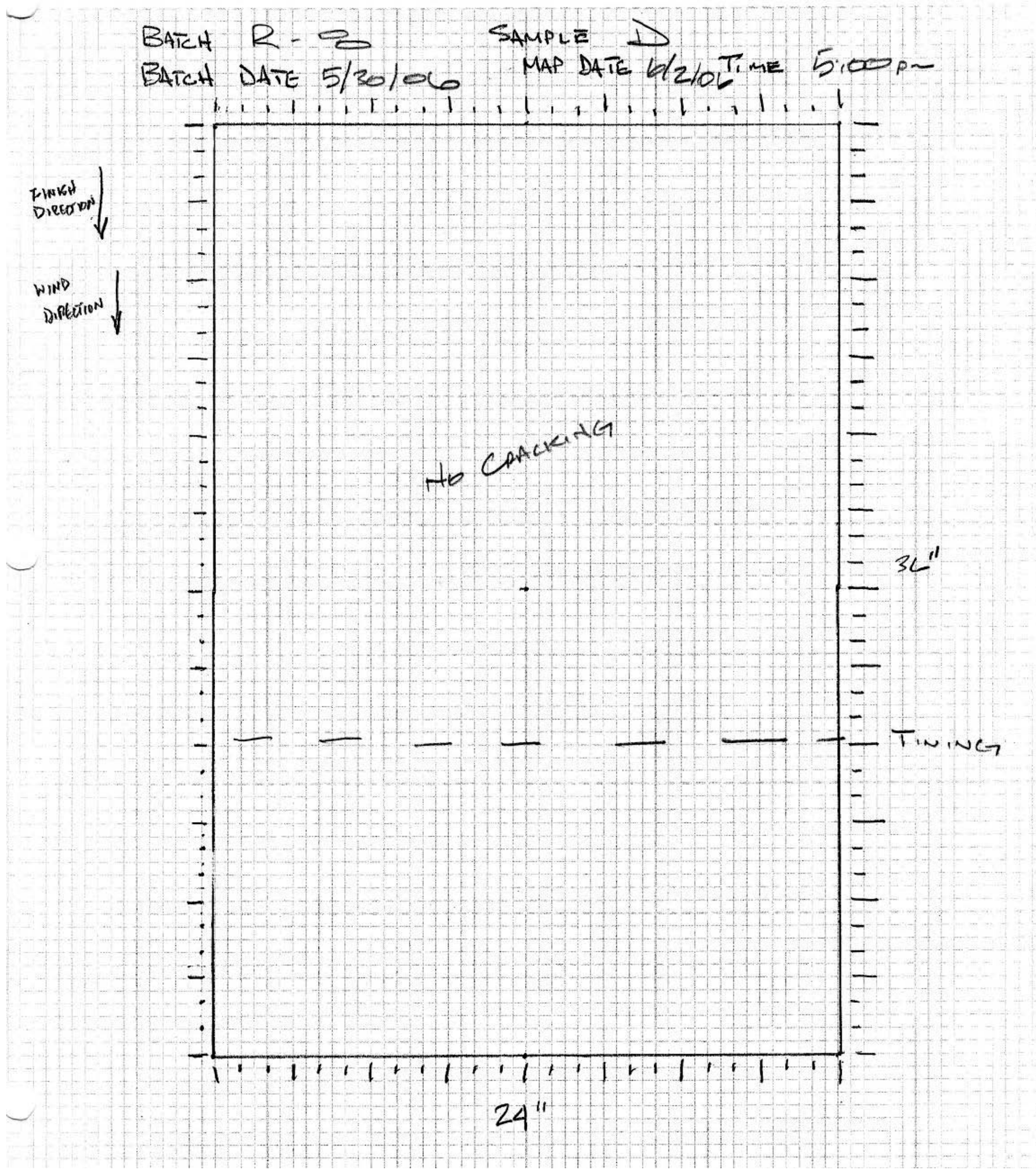


Figure C2.4 Crack Map for Slab R-8D



C.3 Thin Slab Study Batch CC-1 Data

Table C3.1 Batch CC-1 Fresh Properties and Compressive Strength Data

Mix ID: **CC-1**

DATE BATCHED	6/5/06			
TIME OF BATCH	10:08 - Tests @ 10:22			
DESCRIPTION	Retarded 4 fl oz/cwt Thin Slabs CC-Rates			
	Control	125	200	300
Slab ID	A	B	C	D
Time Placed	11:10 AM	11:12 AM	11:19 AM	11:27 AM
Time Screeded	11:15 AM	11:19 AM	11:25 AM	11:32 AM
Time Finished	11:26 AM	11:32 AM	11:34 AM	11:35 AM
Time Tined	X	11:35 AM	11:36 AM	11:38 AM
Time CC 1	X	2:00 PM	2:04 PM	2:08 PM
Time CC 2	X	2:26 PM	2:30 PM	2:34 PM
Time of Wind Start	11:40 AM	11:40 AM	11:40 AM	11:40 AM
Time of First Crack		1:56 PM		
Flow Table		Cone Base	101mm	
Measurement Time	Reading 1	Reading 2	Reading 3	Reading 4
11:26 AM	190	195	195	195
11:47 AM	190	190	190	190
CC-1	DAY 1	DAY 7	DAY 28	
DATE	6/6/06	6/13/06	7/4/2006	
TIME	9:45 AM	1:30 PM		
	psi	psi	psi	
	pounds	pounds	pounds	
1	179	4553	5690	
	2250	57220	71500	
2	365*	4752	6132	
	4580	59720	77060	
3	417*	4543	6626	
	5240	57090	83270	
Notes:	*1:02 PM			

Table C3.2 Batch CC-1 Curing Conditions Data

Job	CC-1					
Date	**6/5/2006**		**Start Time**		**10:20 AM**	**Batch Time**
Date	**Time**	**Air Temp**	**Wind**	**RH**	**Conc. Temp**	**Notes**
6/5/2006	11:30 AM	77.7	0	63	72	Placed Slabs 11:10-11:38
	11:40 AM	77.7	10	63	72	Fans on
	12:00 PM	79	10	62	73	Heat on
	12:20 PM	86.2	10	53	75	Sheen Still on Slabs
	12:45 PM	88.7	10	48	75	Sheen Still on Slabs
	1:08 PM	93.9	10	45	75	Sheen Still on Slabs
	1:37 PM	95	10	44	79	Sheen Still on Slabs
	1:55 PM	97.5	10	42	80	Cracl in Slab B
	2:09 PM	99.7	10	34	80	Applied CC-1 Fan off during Application
	2:34 PM	91.6	10	40	80	Applied CC-2 Fan off during application
	3:06 PM	100	10	38	90	
	3:40 PM	99.4	10	38	90	Cracks in Slab A
	4:45 PM	100.2	10	35	98	
	5:40 PM	100.4	0	34	99	Heat and fans off
						No Cracks in C or D
6/6/2006	8:45 AM	79	10	61	78	ON
	9:35 AM	93.9	10	45	78	
	11:16 AM	95.2	10	43	78	Small Cracks on C and D
	12:53 PM	96.6	10	41	96	H & F off for crack mapping
	1:57 PM	81.7	0	56	87	Finished Mapping (H&F inadvertantly left off)
	2:51 PM	85.8	10	51	87	H & F On
	3:55 PM	91.9	10	46	91	
	4:55 PM	92.5	10	45	91	Off
6/7/2006	8:25 AM	76	10	62	77	On
	9:34 AM	96.1	10	43	91	
	11:10 AM	99.7	10	38	97	
	12:25 PM	101.3	10	37	99	
	1:30 PM	103.8	10	35	101	
	2:31 PM	100.2	10	35	101	Off - No New Cracks
6/8/2006	8:40 AM	77.2	10	57	78	H & F On
	9:00 AM	91.6	10	42	88	
	10:00 AM	95.7	10	39	92	
	11:00 AM	97.5	10	35	94	
	12:00 PM	99.1	10	31	99	
	1:10 PM	100.2	10	31	100	
	3:00 PM	102.9	10	31	101	H & F off - Mapped Cracks

Figure C3.1 Crack Map for Slab CC-1A

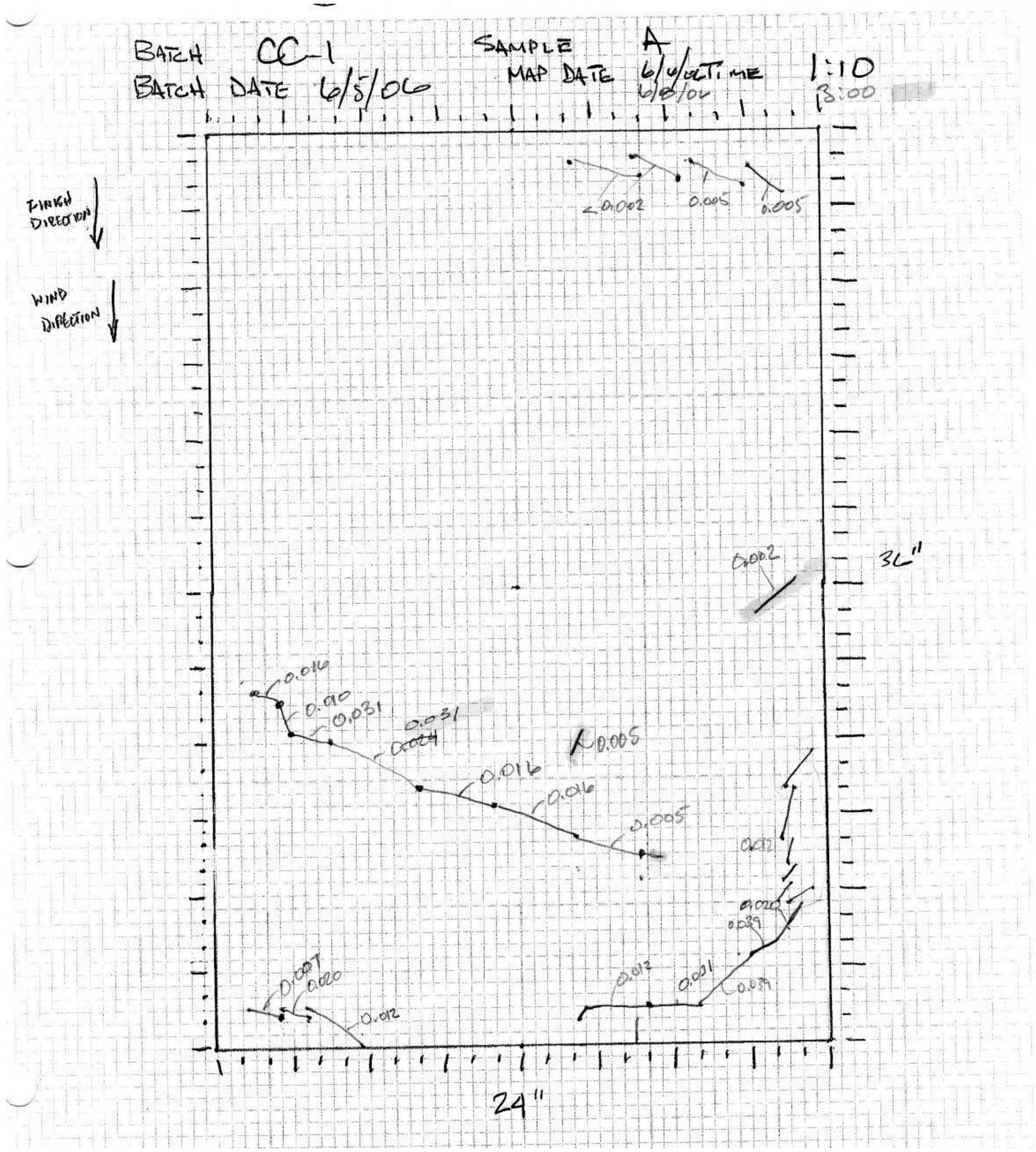


Figure C3.2 Crack Map for Slab CC-1B

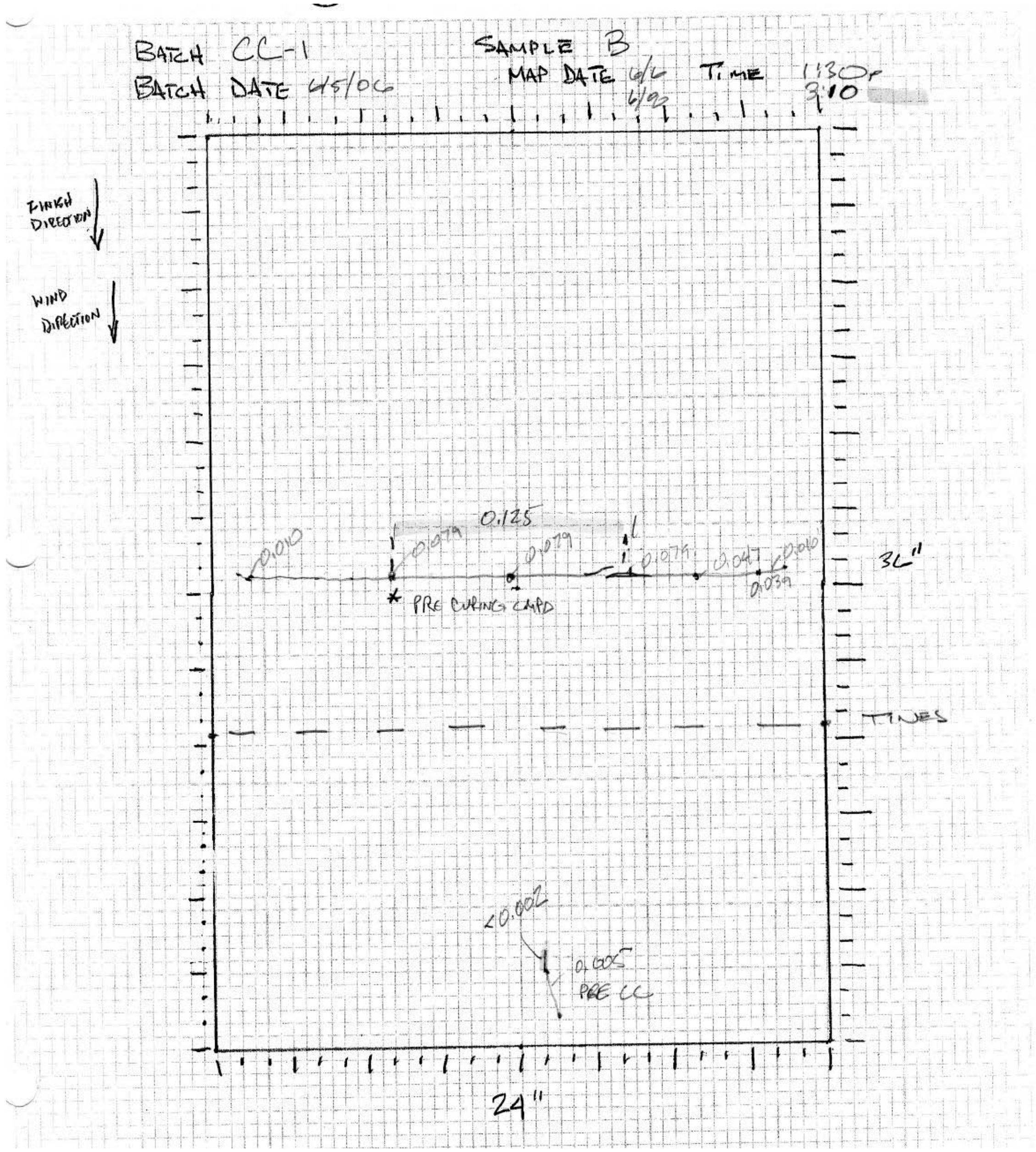


Figure C3.3 Crack Map for Slab CC-1C

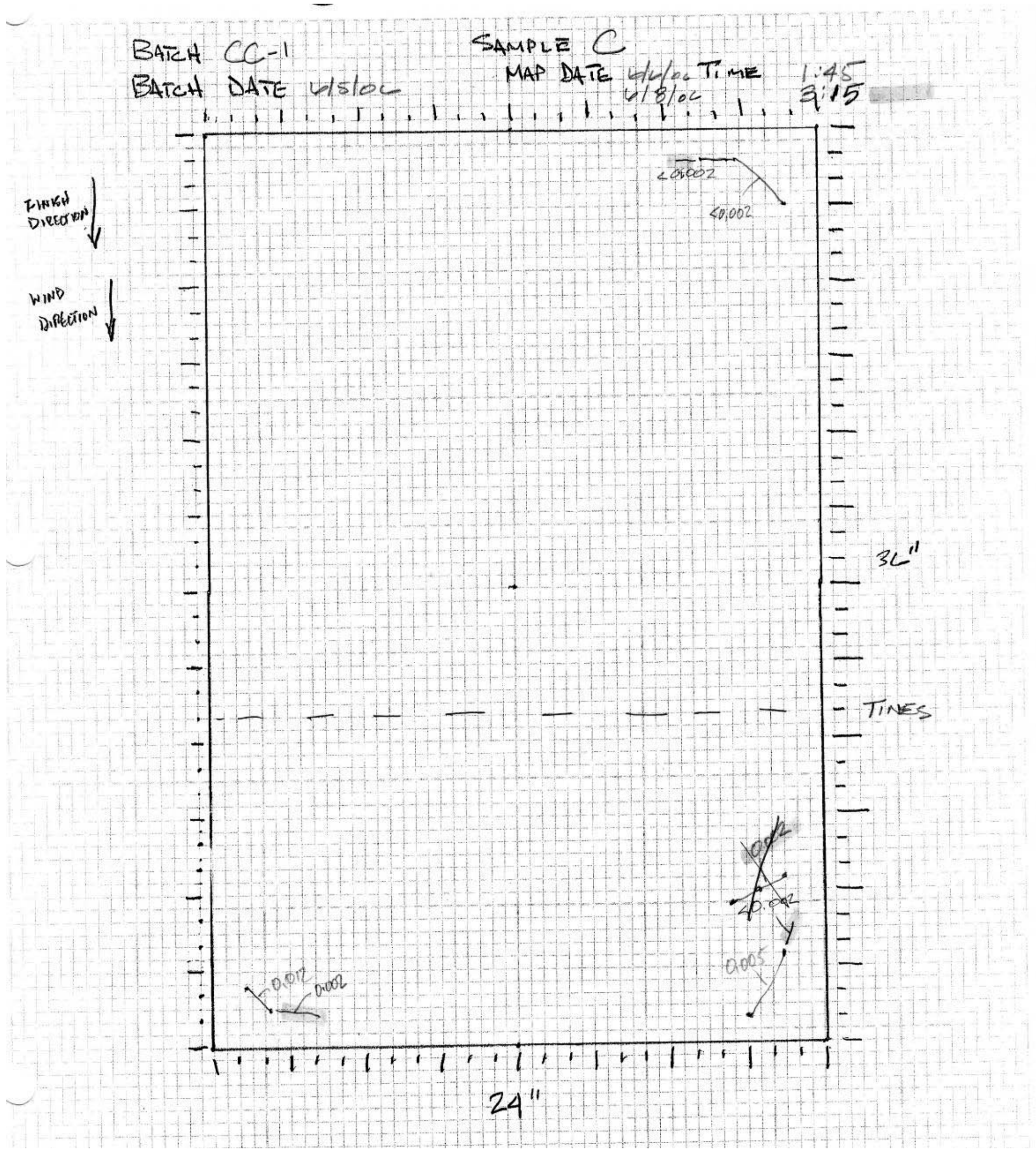
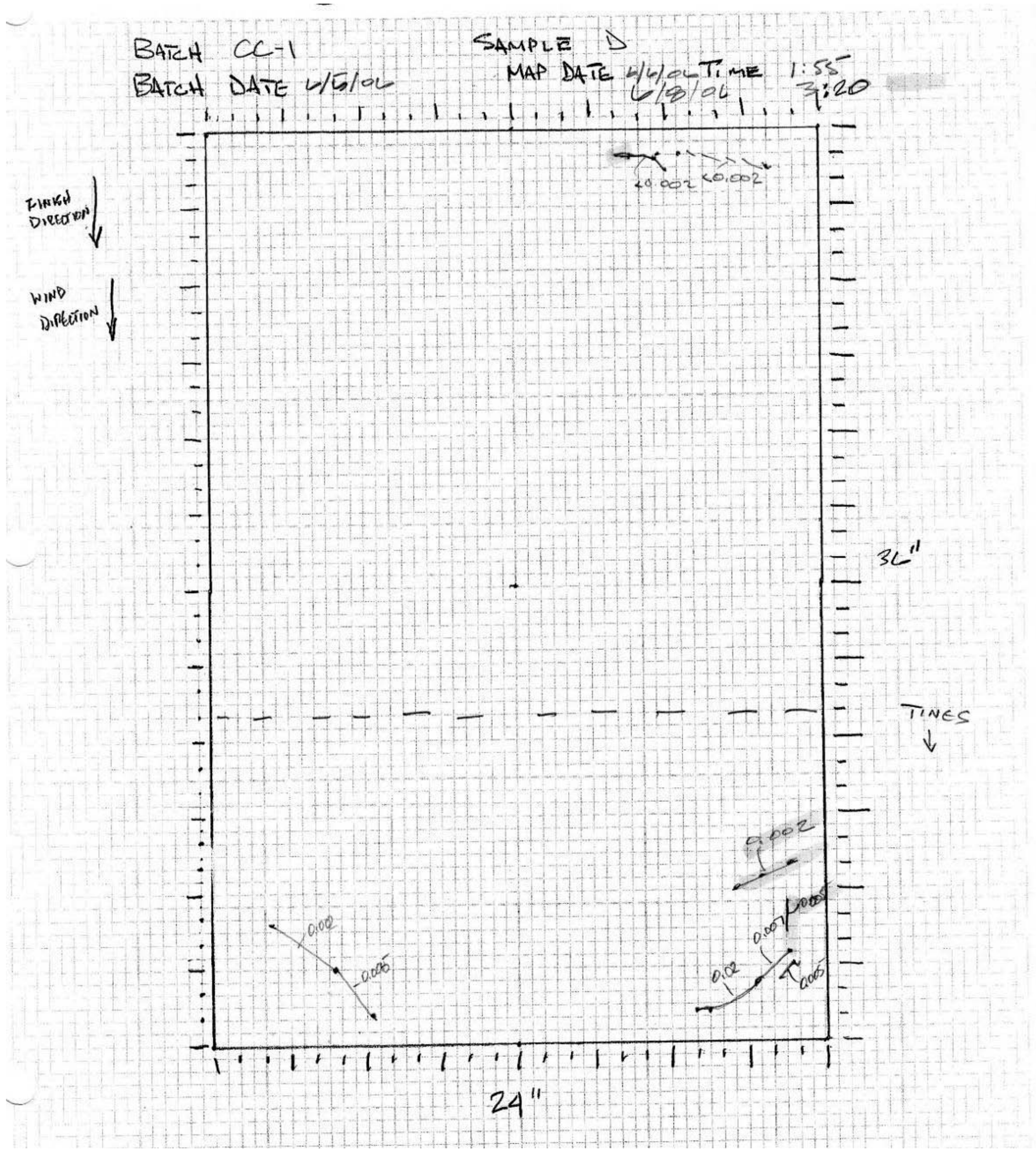


Figure C3.4 Crack Map for Slab CC-1D



C.4 Thin Slab Study Batch CC-2 Data

Table C4.1 Batch CC-2 Fresh Properties and Compressive Strength Data

Mix ID:	**CC-2**			
DATE BATCHED	6/13/06			
TIME OF BATCH	10:00			
DESCRIPTION	Retarded 4 fl oz/cwt Thin Slabs 3 CC-Rates			
	Control	125	200	300
Slab ID	A	B	C	D
Time Placed	10:50	10:52	10:52	11:15
Time Screeded	10:54	11:05	11:10	11:20
Time Finished	11:25	11:28	11:30	11:32
Time Tined	11:34	11:35	11:36	11:37
Time CC 1	X	12:10	12:15	12:20
Time CC 2	X	12:52	12:56	1:00
Time of Wind Start	11:40	11:40	11:40	11:40
Time of First Crack	2:05	6/14/06 8:35	6/14/06 8:35	6/14/06 8:35
Flow Table		Cone Base	101mm	
Measurement Time	Reading 1	Reading 2	Reading 3	Reading 4
11:10 AM	170	180	175	170
11:24 AM	172	171	177	173

CC-2	DAY 1	DAY 7	DAY 28
DATE	6/13/06	6/20/06	7/11/06
TIME	9:00am	10:21	11:45
	psi	psi	psi
	pounds	pounds	pounds
1	150	4453	5609
	1890	55960	70480
2	68	4647	5646
	860	58400	70950
3	72	4496	5531
	910	56500	69500

Table C4.2 Batch CC-2 Curing Conditions Data

Job	**CC-2**					
Date	**6/13/2006**		**Start Time**		**10:00 AM**	**Batch Time**
Date	**Time**	**Air Temp**	**Wind**	**RH**	**Conc. Temp**	**Notes**
6/13/2006	11:40 AM	78.6	10	53	76	H & F On
	12:10 PM		0			H & F Off CC Application 1
	12:20 PM	84.2	10	45	74	H & F On
	12:52 PM	82.2	0	48		H & F Off CC Application 2
	1:02 PM	83.5	10	47		H & F On Closed Enclosure
	2:05 PM	90.7	10	37	90	
	3:09 AM	97.5	10	31	98	
	4:07 PM	101.3	10	27	99	
	5:07 AM	96.8	10	28	100	
	5:40 PM	97.5	10	30	100	H & F Off
6/14/2006	8:35 AM	77	0	51	80	H & F On All slabs have cracks
	9:35 AM	88	10	40	90	
	10:56 AM	93.4	10	39	98	
	12:18 PM	105.6 *	10	30	99	*Therm in front of Heater./
	1:06 PM	99.7	10	28	100	H & F Off Mapped Cracks
	2:08 PM	87.6	0	34	92	H & F On
	3:08 PM	101.1	10	25	101	
	4:02 PM	102	10	26	102	H & F Off
6/15/2006	8:10 AM	78.6	0	53	81	H & F On
	9:40 AM	96.6	10	31	96	
	10:20 AM	97.7	10	31	98	
	11:20 AM	99.7	10	31	101	
	12:56 PM	101.1	10	30	103	
	2:19 PM	103.8	10	30	103	H & F Off
6/16/2006	8:10 AM	81.5	0	58	83	H & F On - No New Cracks
	9:20 AM	97.5	10	36	96	
	10:10 AM	99.7	10	33	100	
	11:20 AM	99.7	10	32	101	
	12:10 PM	100.4	10	32	102	
	1:10 PM	102	10	32	102	
	2:10 PM	102.9	10	33	103	H & F Off Mapped Cracks
	10:20 AM	97.7	10	31	98	

Figure C4.1 Crack Map for Slab CC-2A

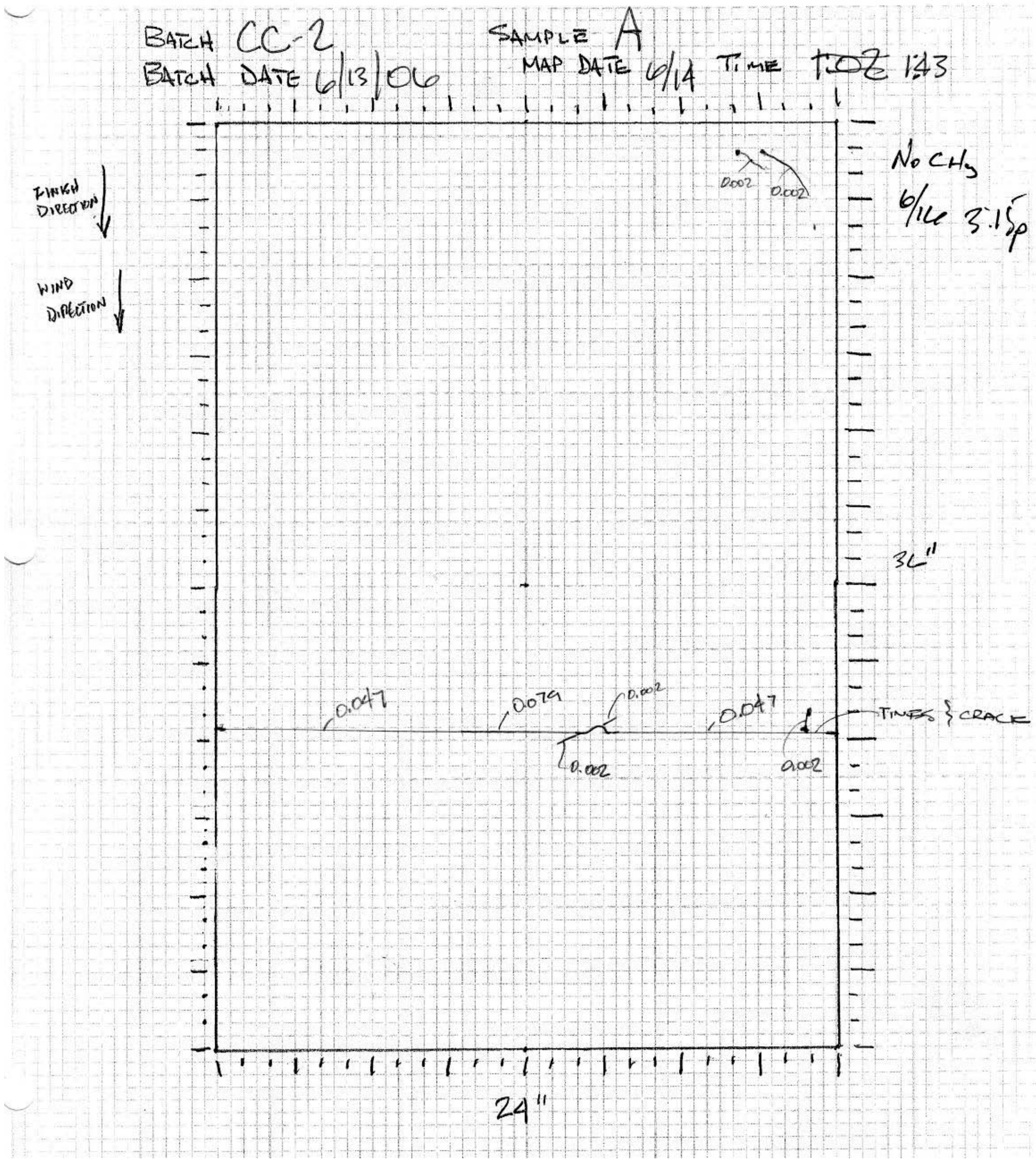


Figure C4.2 Crack Map for Slab CC-2B

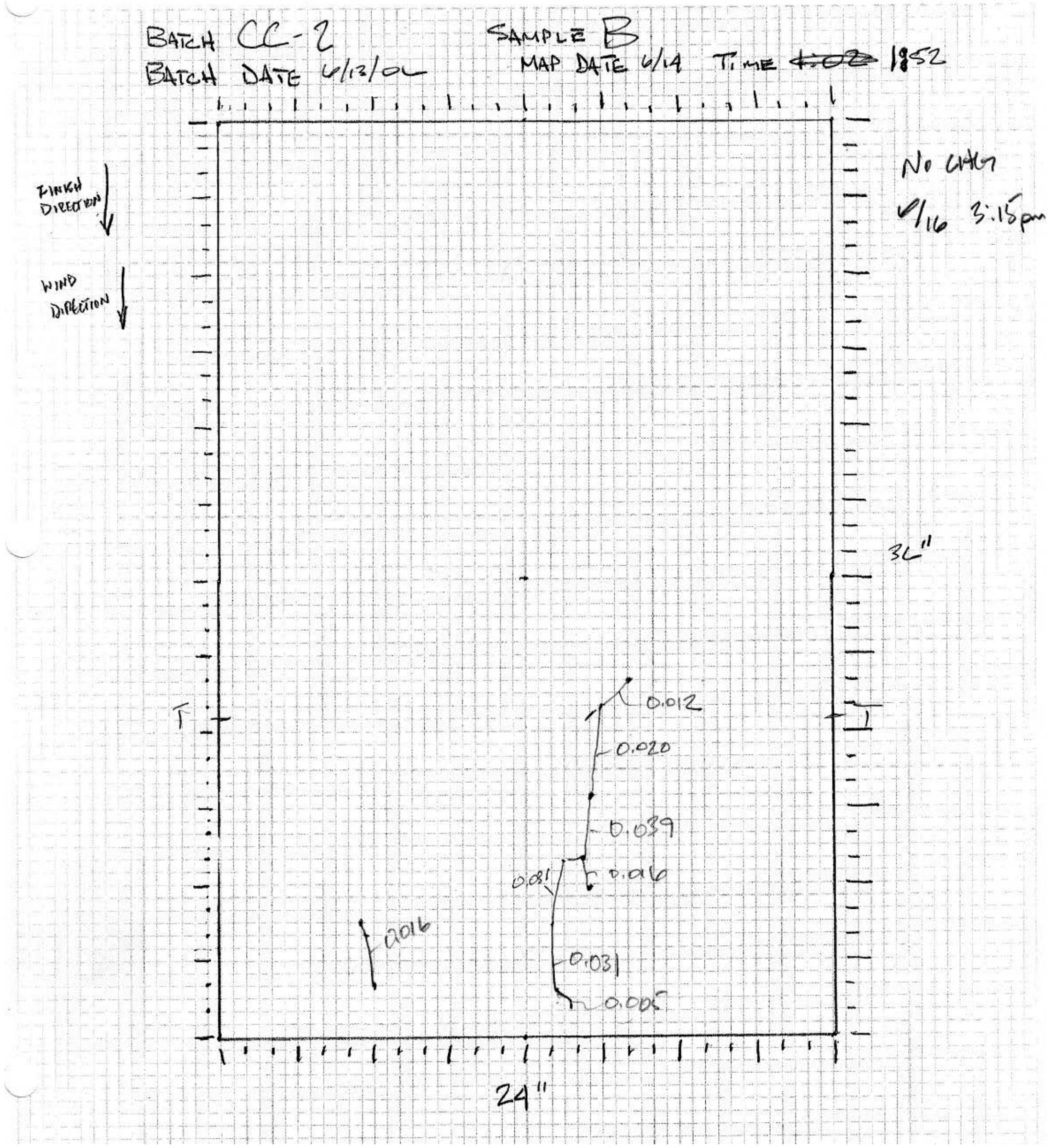


Figure C4.3 Crack Map for Slab CC-2C

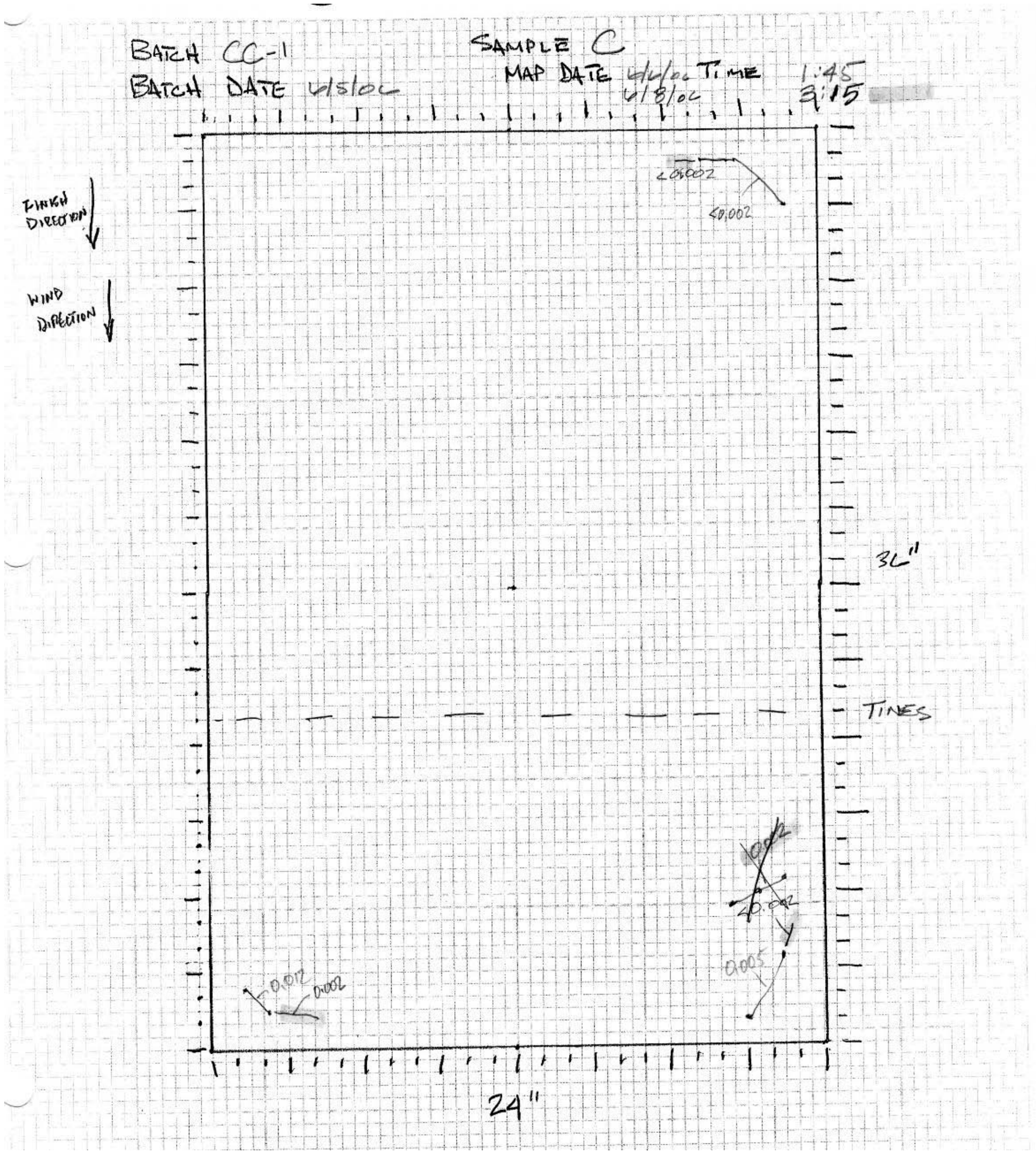


Figure C4.4 Crack Map for Slab CC-2D