Longitudinal Cracking in Concrete at Bridge Deck Dams on Structural Rehabilitation Projects

FINAL PROJECT REPORT

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16. Abstract

The main objective of this project was to identify the causes of longitudinal cracking in newly placed concrete deck segments adjacent to bridge deck expansion dam rehabilitations within District 3-0 of the Pennsylvania Department of Transportation (PennDOT). This objective was accomplished through three tasks. (1) A literature review of the potential causes of early-age cracking in restrained concrete elements, including bridge deck dams and concrete repair sections, was conducted. This task also included a survey of bridge engineers in other PennDOT districts and several other state DOTs and municipalities with regard to concrete bridge deck rehabilitation operations and occurrence of concrete early-age cracking. (2) A review of current PennDOT specifications related to bridge deck construction and rehabilitation was undertaken. This task compared current PennDOT requirements regarding concrete materials, structural/ reinforcement design, and construction operations with the recommendations from the literature review and survey of other transportation agencies and, when needed, suggested modifications to the current PennDOT specifications. In addition, three past and two active bridge deck rehabilitation projects within PennDOT Districts 2-0 and 3-0 were reviewed and inspected to evaluate their compliance with existing PennDOT specifications and literature recommendations to eliminate early-age cracking. (3) A comprehensive experimental evaluation of the material properties of three concrete mixtures commonly used for PennDOT bridge deck projects was performed to evaluate the early- and long-term performance and the risk of cracking of these mixtures. The three mixtures included AAA, HPC, and AAA-P. The following main conclusions were drawn from Tasks 1, 2, and 3 of the project. (a) The most likely causes of early cracking observed in deck rehabilitation projects are inadequate moist curing and failure to properly eliminate plastic shrinkage cracking during construction. In several occasions, it was found that the existing PennDOT specifications for proper water curing of concrete and monitoring of ambient conditions to minimize the evaporation rate from the surface of fresh concrete were not correctly followed. (b) The review of the three past deck rehabilitation projects show that the design of shrinkage and temperature steel reinforcement had been adequate and should not result in early-age cracking. (c) The existing PennDOT concrete mixtures AAA, HPC, and AAA-P can yield adequate performance in the field, provided that they are placed, consolidated, and cured properly. (d) A number of suggested modifications to the current PennDOT specifications are included in this report to minimize the risk of early-age cracking in concrete bridge deck construction and rehabilitation projects.

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Executive Summary

The main objective of this project was to identify the causes of longitudinal cracking in newly placed concrete deck segments adjacent to bridge deck dam rehabilitations within District 3-0 of the Pennsylvania Department of Transportation (PennDOT). This objective was accomplished through three tasks, which are described in detail in this report. **Task 1** (chapter 1) provides a literature review of the potential causes of early-age cracking in restrained concrete elements, including bridge decks and concrete repair sections. This task also provides the results of a survey/ questionnaire of bridge engineers in other PennDOT districts as well as several state DOTs and municipalities with regard to concrete bridge deck rehabilitation operations and early-age cracking. The task concludes with a matrix showing how concrete proportions and material properties, construction practices, and structural design factors affect the early-age cracking tendency of concrete elements. In particular, it is reported that: **(1)** excessive cement content, slump, and compressive strength of concrete contribute to a higher risk of early-age cracking; **(2)** the risk of plastic shrinkage cracking must be eliminated by implementing adequate procedures to minimize water evaporation from the surface of fresh concrete during construction; **(3)** to reduce the risk of cracking, proper water curing methods and sufficient duration (minimum of 7 days) is required; and **(4)** low cover thickness, large rebar sizes, and large rebar spacing also contribute to higher cracking risk.

Task 2 (chapter 2) presents the results of a review of current PennDOT specifications related to bridge deck construction and rehabilitation. This task compares the current PennDOT requirements regarding concrete materials and construction operations with the recommendations from the literature review and survey of other transportation agencies and, when needed, suggests modifications to the current PennDOT specifications. The results show that: **(1)** the allowable cement factors in current PennDOT specifications are excessive and can contribute to early-age cracking; **(2)** the maximum allowable design slump should be limited to 4 inches; **(3)** a maximum allowable 28-day compressive strength must be adopted to prevent the use of excessively strong and stiff concretes that are prone to early-age cracking; **(4)** PennDOT must strictly enforce the requirements on allowable water evaporation rate from the surface of fresh concrete to eliminate the risk of plastic shrinkage cracking; **(5)** provisions must be included to ensure that the surface of newly placed concrete is never exposed to drying for extended

duration; and **(6)** specifications for curing of bridge approach slabs should be modified to require at least 7 days of water curing.

Task 2 also provide a review of the design and construction documentations associated with three past and two active bridge rehabilitation projects within PennDOT Districts 2-0 and 3-0. The concrete mixture design, steel reinforcing bar design, and construction practices implemented by contractors are reviewed against PennDOT requirements, structural design requirements, and literature recommendations. The major findings are as follows. **(1)** Concretes with unnecessarily high (up to 27% higher than required) compressive strengths have been used. **(2)** PennDOT specifications regarding prevention of plastic shrinkage cracking by minimizing the evaporation rate of water from the surface of newly placed concrete were not accurately followed. In one case, it was observed that the finished concrete surface remained totally exposed for 30 to 40 minutes past final finishing activities without application of the mandatory intermediate curing agent. PennDOT must make sure that the air temperature, humidity, and wind speed, as well as concrete temperature are regularly monitored during construction, and proper remediation techniques must be readily available at the construction site if the evaporation rate exceeds 0.1 lb/ft^2hr. **(3)** PennDOT specifications regarding 14-day water curing of bridge decks using continuously wetted double-layer burlaps were not accurately followed by the contractor. Specifically, the curing period was 7 days and the burlap covers were not properly kept wet. **(4)** It is unlikely that the cracking observed in the newly constructed concrete deck areas is a result of inadequate design of steel reinforcement. The review of structural design of reinforcements suggests that the three past projects reviewed have been properly designed with respect to the temperature and shrinkage steel requirements.

Task 3 (chapter 3) reports the results of a comprehensive experimental evaluation of the material properties, performance, and cracking risk of three concrete mixtures commonly used by PennDOT for bridge deck projects. These include AAA, HPC, and AAA-P mixtures. The material properties evaluated include fresh properties (slump and plastic air content), mechanical properties (compressive strength, splitting tensile strength, flexural strength, and elastic modulus), shrinkage and temperature properties (heat of hydration, coefficient of thermal expansion, autogenous shrinkage, drying shrinkage, and restrained ring shrinkage), and durability properties (rapid chloride permeability). The major findings are provided below.

(1) The 28-day compressive strength of these mixtures exceeds the required strength value of 4500psi by up to 35%. **(2)** Other mechanical, thermal and shrinkage properties of the mixtures are considered acceptable in comparison with values reported in the literature. **(3)** Deterministic calculation of the risk of cracking shows a higher risk of cracking for AAA mixture (in comparison with HPC and AAA-P mixtures) due its higher drying shrinkage and higher coefficient of thermal expansion. The restrained ring shrinkage test confirms this conclusion.

The overall results show that the existing PennDOT concrete mixtures can yield adequate performance in the field, provided that they are placed, consolidated, and cured properly. **Based on this research, the most likely causes of early-age cracking observed in the concrete next to the newly installed bridge deck dams are inadequate moist curing and failure to properly eliminate the risk of plastic shrinkage cracking during construction.** These underline the significance of ensuring that contractors carefully comply with PennDOT specifications regarding prevention of plastic shrinkage cracking and proper methods and duration for water curing of concrete bridge decks.

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CHAPTER 1 (TASK 1)

Literature Review and Survey of Transportation Agencies Regarding Causes of Longitudinal Cracking in Concrete Bridge Deck Repair Sections

1.1.0 LITERATURE REVIEW

Cracking of newly placed cement concrete adjacent to bridge deck dam replacements has been observed on several newly rehabilitated sections of bridge decks in the Commonwealth of Pennsylvania (Figure 1-1). This literature review summarizes the potential causes of early-age cracking in concrete decks, which also includes cracking associated with bridge dam replacements and other types of concrete repair sections. Transverse cracking in newly constructed concrete bridge decks has been a common problem reported by many state departments of transportation (DOTs) as well as several cities. Several state DOTs have performed or funded studies over the last few decades to identify the causes and effective mitigation practices for this problem. These studies evaluated typical causes of early-age cracking and the contribution of concrete material properties, construction practices, and structural design factors to this problem.

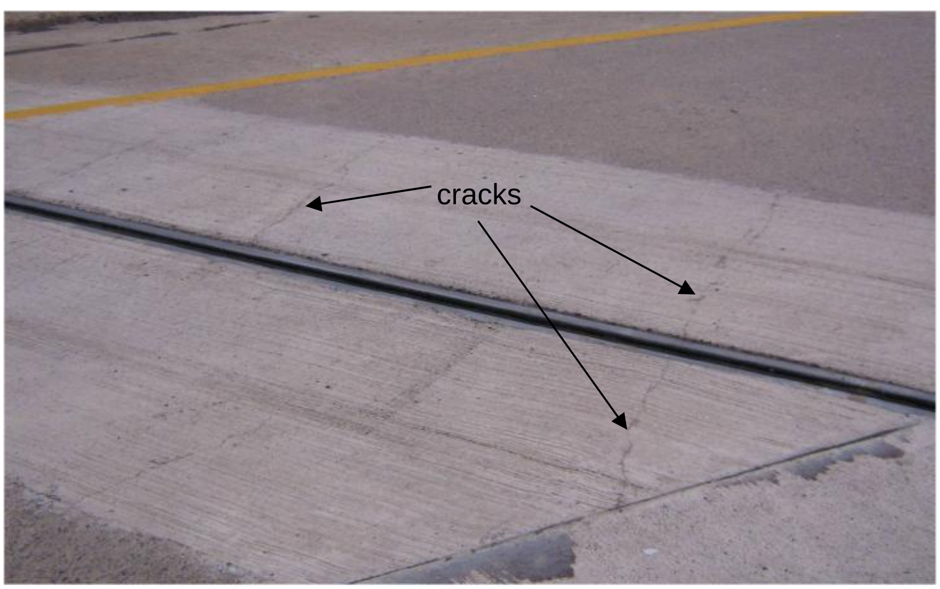

Figure 1-1: Longitudinal and diagonal cracks in a newly placed concrete patch adjacent to bridge deck's expansion dam (photo courtesy of PennDOT District 3-0)

In comparison, past research on cracking in concrete repair sections (including longitudinal cracking in newly placed concrete next to bridge dams) has been considerably more limited. However, it is known that the primary challenge that leads to cracking and poor performance of repair sections is their dimensional instabilitity relative to the substrate (Emmons 1993, Emberson and Mays 1990, Poston et al. 2001). The concrete in the newly placed repair section is prone to volume changes due to plastic, drying, and autogeneous shrinkage and thermal contraction. As this volume change is restrained by adjacent structural components (e.g., the underlying diaphragm beam, and the adjacent deck concrete), transverse tensile stresses develop in the young concrete, which can lead to longitudinal cracking.

In addition, cracking in concrete repair sections can arise from lack of compatibility with the substrate concrete (e.g., concrete diaphragm beam in this case), exessive shrinkage, and/or poor construction-related quality control (including lack of proper and timely curing) (Decter and Keeley 1997, Parameswaram 2004, Morgan 1996). Examples of compatibility problems include differences in elastic modulus or coefficient of thermal expansion, which result in inconsistent deformations between the repair and old concretes when exposed to mechanical loads or changes in ambient environment. Such inconsistent deformations lead to stress development and cracking.

Compatibility requirements of repair patch materials are outlined in Table 1-1. Construction-related details contributing to early deterioration of concrete repair materials outlined by Parmeswaran (2004) include inadequate removal of existing/deteriorated concrete, insufficient curing time, unfavorable climate changes during repair, and insufficient consolidation. Urgency of repair can be a limiting factor in construction repairs.

The wide variety of repair materials available to design engineers can be classified into three primary groups: cementitious mortars and concretes, polymer-modified cementitious concretes, and epoxy-binder concretes (Emberson and Mays 1990, Cusson and Mailvaganam 1996). Among these, a properly designed, placed, and cured conventional Portland cement concrete remains as one of the most reliable, durable, and cost-effective repair materials (Parameswaran 2004). Latex-modified and other types of polymer-modified concretes can be used to improve bonding of the repair to its substrate.

Table 1-1: Compatibility requirements of patch repair materials relative to the existing concrete substrate (from Emberson and Mays 1990, Parameswaran 2004)

Property	Relationship of repair material (R) to concrete substrate (C)
Strength in compression, tension and flexure	$R \geq C$
Modulus in compression, tension and flexure	$R \sim C$
Poisson's Ratio	Dependent on modulus and type of repair
Coefficient of thermal expansion	$R \sim C$
Adhesion in tension and shear	$R \geq C$
Curing and long term shrinkage	$R \leq C$
Strain capacity	$R \geq C$
Creep and Relaxation	Dependent on whether creep causes desirable or undesirable effects
Fatigue performance	$R \geq C$

Using computer-based analysis of repaired reinforced concrete sections, Yuan (1994) showed that the most important properties of repair materials that determine their resistance against cracking are free shrinkage, tensile strain capacity, creep, and the bond strength between repair and adjacent old concrete. Other studies (Decter and Keeley 1997, Morgan 1996) concluded that restrained shrinkage of concrete repairs has a dominant effect on their risk of cracking.

It is the judgement of the authors of this report that longitudinal cracking in repair sections near bridge deck dams is similar in nature to the transverse cracking of newly constructed bridge decks. In both cases, restrained shrinkage results in tensile stress development and cracking, which is predominantly in the direction perpendicular to the longest dimension of the concrete member (i.e., transverse for full bridge decks, and longitudinal for dam repair patches). The problem could be even more severe for dam repairs due to a higher degree of restraint and especially if rapid-hardening concretes are used. In addition, plastic shrinkage, inadequate curing, and structural design factors can contribute to cracking of both bridge decks and repair sections. More relaxed quality control procedures during construction of repairs could exacerbate the risk of plastic shrinkage cracking.

Deck cracking (both longitudinal and transverse) can be the primary cause of early deterioration of bridge decks, and it has been known to significantly decrease the durability and service-life of bridges. It is the nature of these cracks to facilitate penetration of chlorides and moisture and therefore accelerate corrosion of the reinforcing steel. Aside from structural damage, cracking is also unsightly and the resulting distresses significantly decrease the ride quality of the bridge.

Given the limited availability of the literature that has specifically dealt with longitudinal cracking in deck dam repair sections, and given the similarity of this problem to the early-age cracking of concrete bridge decks, the majority of this literature report summarizes studies that aimed at identifying the general causes and effective mitigation practices for early-age cracking of restrained concrete sections in bridge decks. These studies included field surveys, instrumentation and monitoring of cracked bridges, and experimental testing of concrete materials, as well as finite element modeling of bridges to predict stress and strain development. This report includes a summary of published findings by the following state DOTs: California, Colorado, Illinois, Indiana, Iowa, Kansas, Maryland, Minnesota, Missouri, New Jersey, New York, Ohio, Pennsylvania, Texas, Utah, Virginia, and Wisconsin.

1.1.1 Causes of Early Age Cracking in Concrete

Cracking in concrete bridge decks results when the net internal tensile stress is greater than the tensile strength of concrete. Often, tensile stresses are caused as a result of the restrained shrinkage or thermal contraction of concrete, although cracking may also occur due to mechanical loading (e.g., early opening of a bridge to traffic, overloading, and fatigue at a later age). Figure 1-2 shows how tensile stresses develop as a result of restrained shrinkage and thermal contraction of a newly constructed concrete repair patch. The tensile stresses increase with time as concrete experiences more shrinkage until these stresses exceed the tensile strength of concrete, at which time the concrete cracks (Radlinska et al. 2007). In addition to stresses developed as a result of an external restraint, moisture and temperature gradients in concrete (due to preferential drying or cooling at surfaces) can cause a non-uniform shrinkage strain profile, which results in self-restraint and stress formation within concrete.

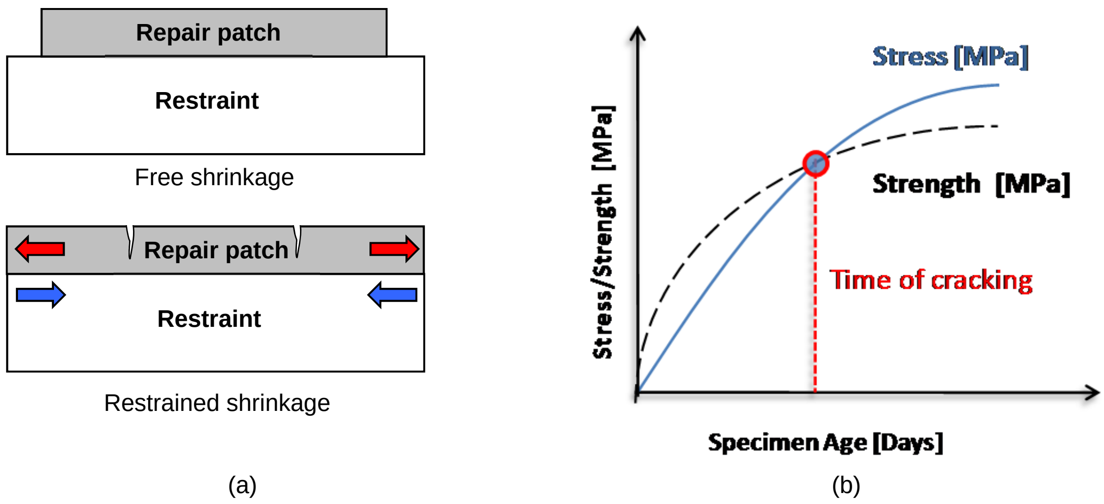

Figure 1-2: (a) Schematic illustration of shrinkage-induced cracking in concrete bridge decks; (b) Time-dependent stress and strength development in concrete leading to early-age cracking

There are several causes or types of shrinkage in concrete: plastic shrinkage, chemical and autogenous shrinkage, drying shrinkage, and carbonation shrinkage.

1.1.1.1 Plastic Shrinkage

When concrete is in a fresh or plastic state, plastic shrinkage cracking can occur if the rate of evaporation exceeds the rate at which the bleed water reaches the surface of concrete (Mindess et al. 2003). As a result, tensile stresses develop at the concrete surface, which due to the very low tensile strength capacity of fresh concrete, can result in cracking (Figure 1-3). In addition, differential settlement over a rebar or where a change in the member's cross section occurs can lead to plastic cracking. Plastic shrinkage cracking has been found to be especially common in high water-to-cementitious ratio (w/cm) concretes as well as high strength mixtures containing silica fume (the latter due to a decrease in bleed water and reduction in pore size, which exacerbates tensile stresses) (Cohen et al. 1990). Plastic shrinkage can be reduced with proper moist curing, reducing evaporation rates, and installing wind breaks, so the surface of the concrete never dries.

Figure 1-3: Plastic shrinkage crack in concrete slab (photo from PCA 2011)

1.1.1.2 Chemical and Autogenous Shrinkage

When cement hydrates, the net volume of hydration products (e.g., C-S-H gel, portlandite, and other products) is less than the volume of the reactants (e.g., cement and water). This volume reduction is known as chemical shrinkage and is approximately equal to 64 ml per 1 kg of Portland cement ($1.77 in^3/lbs$ of cement) for neat cement paste (Jensen and Hansen 2001). As long as concrete is in a plastic state, this chemical shrinkage results in an overall settlement of the upper surface of fresh concrete; however, no tensile stresses are developed. Only after concrete sets, further chemical shrinkage serves as a driving force for autogenous shrinkage, which increases the risk of cracking of restrained concrete members. It should be noted that chemical shrinkage, which is an intrinsic property of concrete, does not lead to cracking unless it results in large autogenous shrinkage.

After concrete sets, chemical shrinkage can no longer be accommodated by settlement. As such, air-filled capillary voids form in the interior of the concrete as the water is consumed by hydration reactions. This phenomenon is commonly known as self-desiccation of concrete (Radlinska et al. 2008), resulting in a uniform drying of the entire cross section of concrete member. This phenomenon is fundamentally different from drying shrinkage, in which drying occurs at exposed surfaces of concrete while the interior of the concrete (i.e., beyond a few inches from surface) remains near saturation.

As a result of self-desiccation and formation of many small air-filled voids that are uniformly distributed throughout the concrete member, capillary stresses develop, which causes a uniform volume reduction of concrete. This volume reduction that resulted from self-desiccation is known as autogenous shrinkage. If the concrete member is restrained, even in the absence of any external drying, autogenous shrinkage can cause tensile stresses and cracking (ACI-231 2009). Autogenous shrinkage is inversely related to the size of the capillary pores inside the concrete. As such, this type of shrinkage can be especially problematic in low w/cm (e.g., <0.36) and high cement content concretes, or when silica fume has been used (Jensen and Hansen 1996). It also increases at higher temperatures (Jensen and Hanson 1999). It should be mentioned that for low w/cm concretes, even proper external moist curing may not fully alleviate the autogenous shrinkage cracking, since the external water may not penetrate more than a few millimeters (≈¼ inch) into the concrete due to low permeability of the concrete matrix. As a result, the core of the concrete member may self-desiccate while the surface remains saturated.

To address this problem, a new methodology has been developed known as internal curing. In this technique, concrete is entrained by fine and well-distributed water reservoirs (e.g., saturated lightweight fine aggregates or super-absorbing polymers), which can gradually release this water to the interior of the concrete to prevent self-desiccation (Bentz and Jensen 2004, Jensen and Hanson 2002, ACI-224R 2001). It should be noted that each saturated light-weight particle can only protect a thin layer (≈¼ inch) of its surrounding concrete from self-desiccation. This is due to limited permeability of the concrete matrix. As such, it is important to use fine and well distributed pre-saturated light-weight aggregates. Course light-weight aggregates and aggregates that are not properly saturated prior to mixing in concrete have been shown to be unable to prevent self-desiccation (Bentz and Snyder 1999).

1.1.1.3 Drying Shrinkage

Drying shrinkage results as moisture is lost from the surface of hardened concrete. Moisture evaporation results in the development of capillary stresses, which reduces the volume of concrete. If this contraction is restrained (Figure 1-4), tensile stresses and cracking can result (similar mechanism as desiccation cracking of clays). In the absence of restraint, no visible cracks would form. Figure 1-5 is a schematic illustration showing that without a restraining friction, concrete slabs would shrink but would not crack. However, in a more realistic scenario, the restraint provided by the sub-base results in cracking of the slab as it undergoes drying shrinkage. For concrete bridge decks, in most applications, external restraint can be caused by girders and stay-in-place (SIP) forms. Aggregate and reinforcing steel can provide internal restraint, which may lead to micro-cracking.

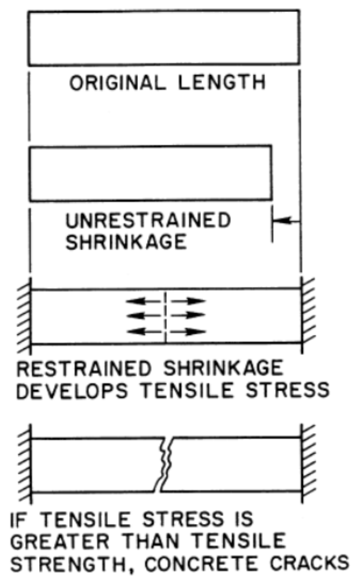

Figure 1-4: Cracking of restrained concrete due to drying shrinkage (ACI 224R 2001)

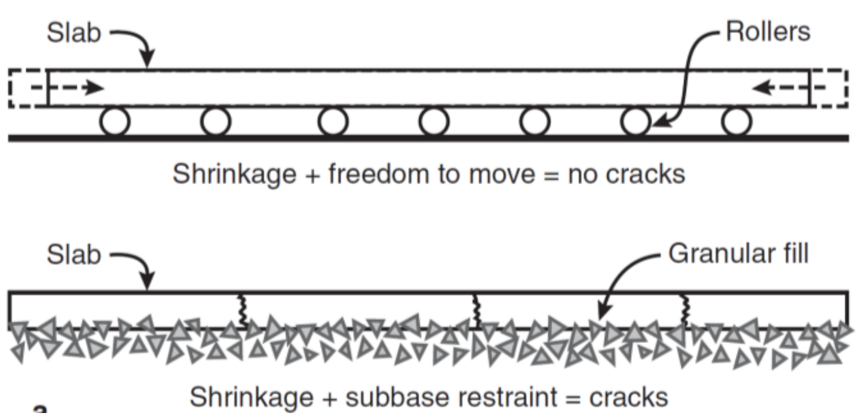

Figure 1-5: Restrained drying shrinkage resulting in cracking of concrete slab (PCA 2011)

Drying shrinkage of concrete is significantly dependent on its aggregate content, w/cm, and the relative humidity of the ambient. Aggregates don't shrink (or shrink very little) compared to cement paste. As such, the presence of aggregates provides an internal resistance and significantly reduces the shrinkage of concrete. Pickett (1956) suggested the following equation

(Equation 1-1) that relates the shrinkage of concrete (ϵ_{con}) (µm/m) to the shrinkage of cement paste (ϵ_p) (µm/m) and the aggregate volume fraction (V_{agg}):

$$\varepsilon_{con} = \varepsilon_p (1 - V_{agg})^n \tag{1-1}$$

where n is a parameter that ranges from 1.2 to 1.7, depending on the stiffness of the aggregates and the paste (L'Hermite 1960). This correlation is shown graphically in Figure 1-6a. In addition, the drying shrinkage of concrete depends on the stiffness of aggregates (Figure 1-6b). Aggregates with low absorption and high modulus of elasticity are the most effective at reducing concrete shrinkage. It should be noted that although generally light-weight aggregates have low stiffness, if they are pre-saturated, they serve as internal water reservoirs and as such, can reduce autogenous shrinkage and reduce or at least delay drying shrinkage.

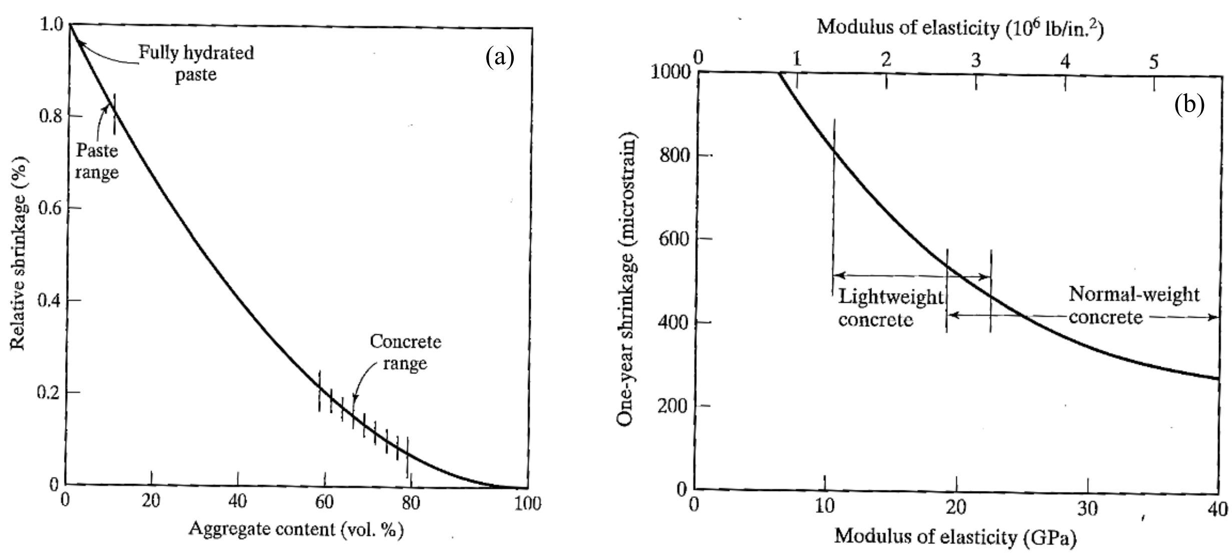

Figure 1-6: (a) Concrete shrinkage as a function of aggregate volume fraction; (b) effect of aggregate stiffness on shrinkage of concrete (Mindess et al. 2003)

Moisture evaporation from concrete is a function of temperature and relative humidity of the ambient. Concrete dries faster and shrinks more in hot and dry ambient conditions. Figure 1-7ab shows the approximate relationship between relative humidity and mass loss and drying shrinkage of concrete.

In addition, w/cm has been shown to affect drying shrinkage, mainly due to faster drying in high w/cm high-porosity concrete and a lower stiffness of these materials. Also, specimen geometry affects the magnitude of drying shrinkage. Specimens with a small surface-to-volume ratio (e.g., large square and cylindrical sections) dry slowly, compared with thin specimens with high surface area (e.g., slabs, decks, overlays). As such, the former shows a slower drying shrinkage over time, although the ultimate shrinkage may be similar.

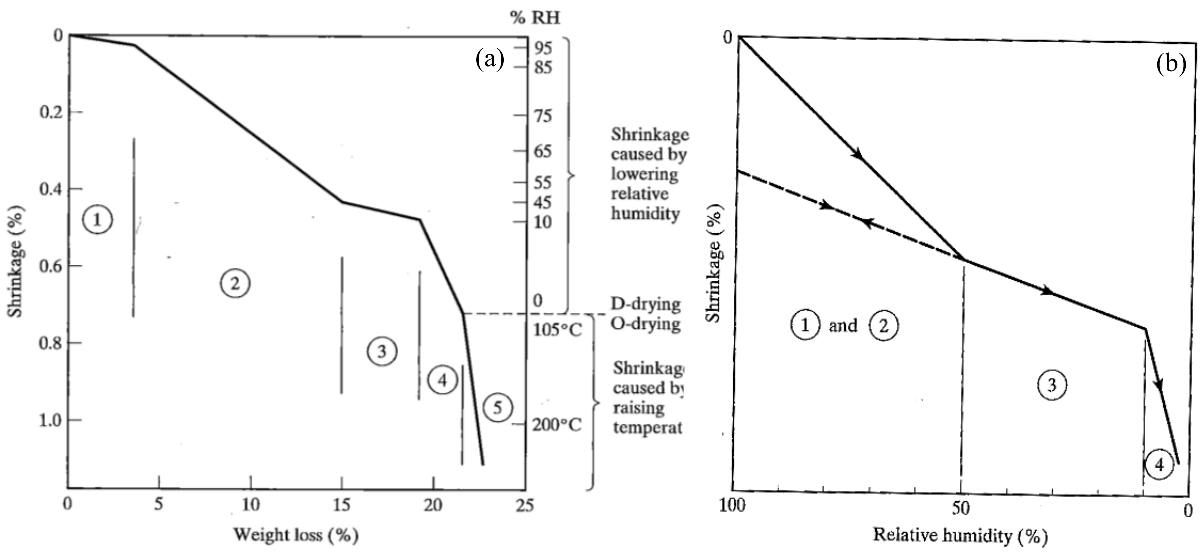

Figure 1-7: Relationship between ambient relative humidity (%RH) and (a) weight loss, and (b) drying shrinkage of concrete (Mindess et al. 2003)

1.1.1.4 Carbonation Shrinkage

In addition to drying shrinkage caused by moisture evaporation from concrete, the atmospheric carbon dioxide (CO_2) can chemically react with hardened cement paste and cause an irreversible shrinkage, known as carbonation shrinkage. The magnitude of carbonation shrinkage is a function of relative humidity and temperature (Mindess et al. 2003). Since atmospheric CO_2 is always present (except in very controlled laboratory chambers), carbonation shrinkage always occurs simultaneously with drying shrinkage. The majority of drying shrinkage measurements performed in laboratories and all field measurements result in reporting shrinkage values that are a combination of drying and carbonation shrinkage.

1.1.1.5 Thermal Contraction

Another source of volume instability and potential for cracking of concrete is thermal contraction, which is especially a concern with early-age concrete. At early ages, the heat of hydration causes the temperature of fresh concrete to rise. Often concrete sets near its peak temperature, and afterwards, as concrete cools, it contracts (ACI-231 2010). If this thermal contraction is restrained by adjacent members (for example, bridge girders, abutments, approach slabs, adjacent existing deck slabs and longitudinal rebar, and/or metal deck pan), tensile stresses develop inside the concrete that can result in cracking. Thermal cracking can occur from both externally applied temperature gradients as well as gradients formed internally. The temperature difference between peak concrete temperature and temperature of supports (e.g., steel forms or girders) provides a source of external temperature gradient. The supports also act as a restraint as concrete cools, resulting in residual tensile stresses and possibly cracking (TRB Circular E-C107: 2006). Internal temperature gradients form when concrete does not cool at the same rate throughout. This occurs typically when a concrete surface is exposed to ambient air temperatures (as such, it cools or heats quickly) while the interior of the concrete remains at a different temperature. Other factors contributing to thermal cracking are aggregate content, cement content, and w/cm. Low aggregate concretes with low w/cm are prone to significant heat of hydration development, which can subsequently result in thermal contraction cracking (ACI-231 2010). Often, to control temperature development of concrete, supplementary cementitious materials (e.g., fly ash or slag) are used. Other means include cooling the concrete ingredients (water, aggregate) prior to mixing.

1.1.1.6 Effect of Mechanical Loads

Previous research (Schmitt and Darwin 1995, Krauss and Rogalla 1996, Frosch et al. 2003, Hadidi and Saadeghvaziri 2005) showed that tensile stresses caused by mechanical loading of the bridge are far smaller than the stresses generated by restrained shrinkage, unless DOT specifications prohibiting the opening of the bridge to traffic or heavy construction equipment before the concrete has gained a minimum strength are not followed. In repair applications, the loads caused by adjacent traffic lanes remaining open during construction may contribute to early-age cracking. Issa (1999) showed that vibrations due to adjacent traffic lanes will only contribute to plastic cracking when concrete is under-vibrated or has too high of slump. In deck construction for continuous multi-span bridges, flexural cracking of concrete often results from negative moments at piers caused by the dead load of concrete that is poured subsequently in the middle of the span (i.e., positive moment area). In order to reduce the moments causing the tensile strain, it is recommended that concrete be placed first in the center of the continuous bridge deck spans at the positive moment regions before the negative moment regions (Babaei and Hawkins, 1987, Issa 1999). Other factors that may result in cracking of concrete in the long term are creep and fatigue. When concrete is subject to sustained loads (e.g., dead load), it continues to deform. As the deformation exceeds the tensile strain capacity, concrete may crack. In addition, repeated loading (e.g., traffic) can cause graduation formation and propagation of microcracks, which leads to surface macrocracks after many cycles of loading.

With respect to early-age cracking, another phenomenon influencing the magnitude of tensile stresses inside concrete is stress relaxation (ACI-224R 2001). Relaxation is an alternative form of creep that is caused by the viscoelastic nature of concrete. Both the solid microstructure as well as the internal moisture of concrete can move gradually in response to sustained stresses, resulting in stress relaxation. This means that the actual magnitude of tensile stress in concrete is less than what is predicted by Hook's law from the magnitude of shrinkage strain. This is shown in Figure 1-8 (Weiss et al. 1998). Research has shown that stress relaxation is lower for concrete with higher elastic modulus; as such, high strength/stiffness concrete is even more prone to shrinkage cracking (Krauss and Rogalla 1996, Darwin et al. 2004).

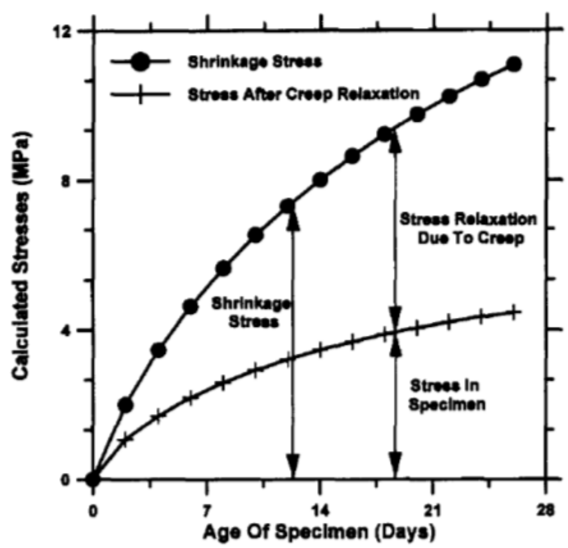

Figure 1-8: Comparison between elastic and relaxed stresses in a restrained concrete slab undergoing shrinkage (from Weiss et al. 1998)

The remainder of this chapter discusses (in more detail) factors that have been found to contribute to or mitigate early-age cracking in restrained concrete sections, especially bridge decks. These factors have been divided into three categories: concrete material properties, structural design factors, and construction practices. Study approaches in previous research included laboratory experiments on concrete samples, full-scale field investigations, and analytical/finite element simulations to evaluate the causes of cracking on bridge decks.

1.1.2 Effect of Concrete Material Properties on Its Early Age Cracking

Concrete material properties have been the subject of most past research for the mitigation of early-age cracking on concrete bridge decks. The following discussion on the role of material properties is divided into (A) the effect of concrete mixture proportions, (B) the effect of concrete ingredients, and (C) the effect of concrete's fresh and hardened properties.

1.1.2.1 Effect of Concrete Mixture Proportions

1.1.2.1.1 Water to Cementitious Materials Ratio (w/cm)

Some past research shows that low w/cm tends to increase early age cracking (Brown et al. 2001). This is thought to be mainly caused by (a) increased heat of hydration and subsequent thermal stress development, (b) increased self-desiccation and autogenous shrinkage, and (c) increased stiffness and reduced stress relaxation, which result in higher magnitudes of stress development. A higher degree of cracking often observed for high strength concrete bridge decks is the result of these phenomena (Darwin et al. 2004). Lower w/cm increases the need for proper moist curing due to lack of bleed water available during hydration of the concrete. In addition to external curing, proper internal curing may be needed to mitigate self-desiccation (Bentz et al. 2005).

On the other hand, it has been suggested that high w/cm can lead to increases in plastic shrinkage and settlement cracking over reinforcement. These concretes also tend to shrink more due to drying and carbonation (Krauss and Rogalla, 1996). Several studies have recommended a reduction in w/cm to reduce cracking (Iowa DOT 1986, Schmitt and Darwin 1995, Ramey et al. 1997, French et al. 1999). Maximum allowable w/cm in the range 0.40 (Kochanski et al. 1990) to 0.48 (PCA 1970) has been suggested. McLeod et al. (2009) suggested a maximum w/cm ranging from 0.42 to 0.45.

1.1.2.1.2 Cement Content

There is a strong positive relationship between concrete cracking and increased cement content (Krauss and Rogalla 1996, Schmitt and Darwin 1999, Hadidi and Saadeghvaziri 2005; also see Figure 1-6(a)). Generally, cement paste is the phase in concrete that undergoes shrinkage while aggregates are volumetrically stable. As a result, a reduction in aggregate content directly impacts the magnitude of shrinkage in concrete (French et al. 1999). In addition, cement paste is the phase that causes evolution of heat of hydration and as such, high cement mixtures are more prone to thermal cracking (ACI-231 2010). High cement content with low w/cm concretes tend to be more susceptible to early-age cracking than low cement concretes with high w/cm (Krauss and Rogalla 1996, Darwin et al. 2004). The maximum recommended cement content to prevent cracking had been reported as 611 to 725 lb/yd^3 of concrete. However, more recently, McLeod et al. (2009) found these recommendations to be too high on cement content and suggested limiting the cement factor to between 500 and 540 lb/yd^3.

1.1.2.1.3 Water Content

ACI 224R-01 (2001) recommends keeping the water content low in concretes in order to avoid excessive drying shrinkage and plastic shrinkage cracking. Schmitt and Darwin (1999) found a significant trend with increased water content and cracking on monolithic bridge decks. On overlays, both very high and very low water contents had shown a positive relationship with bridge deck cracking. Babaei and Purvis (1994) suggested a maximum water content of 323 lb/yd^3.

1.1.2.1.4 Aggregate Content

The same principle of using a lower volume of cement paste applies to using a higher volume of aggregate. Aggregates generally don't shrink and can provide internal restraint to mitigate shrinkage of concrete. Schmitt and Darwin (1995) and Darwin et al. (2004) recommended capping the paste volume (excluding air) around 27%. Higher crack densities were observed on monolithic bridge decks with paste volumes (excluding air) above 27.5%. Krauss and Rogalla (1996) and French et al. (1999) suggested that a reduced paste volume and reduced total water content will decrease shrinkage cracking. Optimization of aggregate packing and particle size distribution to achieve a higher aggregate content has been suggested (Shiltstone 1990, McLeod et al. 2009).

1.1.2.1.5 Air Content

Poppe (1981) and Krauss and Rogalla (1996) did not find a significance to concrete cracking with respect to air content. However, Schmitt and Darwin (1999) observed that increased air contents, especially above 6%, proved to decrease crack density. French et al. (1999) found that higher air content reduced cracking in bridge decks, but data for their study was limited.

<u>1.1.2.2 Effect of Concrete Ingredients</u>

1.1.2.2.1 Cement Type

Type II cement typically reduces cracking due to the lower thermal gradient during the early stages of hydration because of the lower heat of hydration. The lower modulus of elasticity during early age is also thought to mitigate the cracking (Krauss and Rogalla 1996, Brown et al. 2001). Type III cement, on the other hand, may considerably increase cracking due to rapid setting (which may lead to improper consolidation and finishing) and a significant increase in the heat of hydration and autogenous shrinkage. Higher early stiffness also results in lower stress relaxation (Mehta and Monteiro 2006). Finer cements and cements with high sulfate contents will reduce setting time and increase early strength/stiffness and therefore exhibit an increase in crack tendency. Low early-strength concretes made with type II cement should be preferred for bridge deck construction unless "open-early" is an issue (Hadidi and Saadeghvaziri 2005).

Shrinkage-compensating concretes can minimize or eliminate shrinkage cracking. Use of shrinkage-compensating concrete with Type K cement has prevented the formation of early-age cracking in restrained shrinkage testing (Brown et al. 2001). Specimens tested with ettringite-forming expansive additive also helped to reduce cracking. Several transportation agencies have reported that shrinkage-compensating concretes helped to mitigate early-age cracking on bridge decks (ACI 2001, Krauss and Rogalla 1996). Shrinkage-compensating cements attempt to balance autogenous and drying shrinkage with a designed expansion to prevent cracking. As long as proper restraining is provided, expansion of self-stressing cements can pre-stress concrete and improve its tensile capacity (Bentz and Jensen 2004).

1.1.2.2.2 Aggregates Types

Aggregates that are resistant to deformation and cracking, have low shrinkage, high modulus of elasticity, and low absorption perform the best in terms of reducing the ultimate shrinkage of concrete (ACI 2001, Krauss and Rogalla 1996). Babaei and Purvis (1994) recommended maximum absorption capacity of 0.5% for coarse and 1.5% for fine aggregates. McLeod et al. (2009) have proposed the following requirements for normal weight aggregates. For coarse aggregate, maximum absorption should be less than 0.7%, and maximum deleterious substances should be: passing #200 sieve< 2.50%, shale<0.50%, clay lumps and friable particles<1.00%, coal<0.50%. For fine aggregates, maximum deleterious substances should be: passing #200 sieve< 2.00%, shale<0.50%, clay lumps and friable particles<1.00%.

As mentioned earlier, light-weight aggregates have higher porosity and generally a lower stiffness than normal-weight aggregates. However, when they are properly pre-soaked before mixing into concrete, they serve as internal water reservoirs and as such, can reduce autogenous shrinkage and reduce or at least delay drying shrinkage.

1.1.2.2.3 Mineral Admixtures

Mineral admixtures are often used in concrete as partial replacement for Portland cement. The uses of fly ash, ground granulated blast furnace slag, and silica fume have advantages for increasing the long-term strength and durability of concrete due to the pozzolanic reaction. Previous research studied the effect of these mineral admixtures on the cracking tendency of concrete.

In most studies, concrete mixtures containing silica fume were associated with increased cracking. Silica fume can increase the potential for cracking by both plastic shrinkage (due to lack of bleed water) and autogenous shrinkage (due to pore size reduction) (Cohen et al. 1990, Mindess et al. 2003, Bentz and Jensen 2004). The increased early-age strength and stiffness can also cause less stress relaxation. The impact on ultimate drying shrinkage is often insignificant (PCA 2011). Schmitt and Darwin (1999) observed an increased crack density on bridge decks most likely due to the lack of bleed water in silica fume mixtures (also containing water reducing admixtures). Krauss and Rogalla (1996) suggested that early-age cracking in silica fume concrete mixtures could be attributed to early higher elastic modulus and lower creep. Some literature has suggested that silica fume can provide a decrease in cracking if careful and proper curing procedures are used (Ozyildirim 1991).

Riding et al. (2008) researched four different fly ash mixtures to determine their effect on early-age cracking in concrete. It was found that even though tensile strength gain was retarded, cracking was reduced due to lower thermal strains. It was concluded that the combination of lower heat of hydration, higher creep, and lower elastic modulus development improves the crack resistance of concrete. Drying shrinkage has been found unaffected by the addition of fly ash (PCA 2011); however, reduction in mass transport rate due to the use of fly ash can reduce the rate of drying and carbonation of concrete.

Lura et al. (2001) and Lee et al. (2006) researched the effects of ground granulated blast-furnace slag (GGBFS) used as Portland cement replacement in concretes. Although the former study was concerned with the effect of temperatures on the early-age shrinkage of concrete, both studies observed higher autogenous shrinkage in concretes with GGBFS replacement. These studies could not conclusively determine the effects of GGBFS on the tendency of early-age cracking.

1.1.2.2.4 Chemical Admixtures

Studies on chemical admixtures included both field observation of bridge decks and laboratory experiments. Some showed an inconclusive effect of set-modifying admixtures on the cracking tendency of concrete bridge decks (Schmitt and Darwin 1999, ACI 224R-01 2001). Others (Krauss and Rogalla 1996) found that retarders lower the heat of hydration, which results in a decrease in thermal cracking. At the same time, retarders also delay setting, which leaves the concrete susceptible to plastic cracking. In a lab experiment (Krauss and Rogalla 1996), specimens containing accelerators cracked 4 days earlier than the control specimens. In general, accelerators can increase shrinkage, early temperature rise, and early modulus of elasticity, all of which tend to increase the tendency of early-age cracking. McLeod et al. (2009) discouraged the use of any set-modifying admixture for development of low-cracking concrete for bridge decks.

Shrinkage reducing admixtures (SRA) are used to reduce the amount of both autogenous and drying shrinkage of a concrete mixture. SRA reduces shrinkage by up to 50% by reducing the surface tension of concrete's pore solution (Radlinska et al. 2008, Rajabipour et al. 2008, Weiss et al. 2008). By studying restrained concrete slabs, Weiss et al. (1998) showed that the use of SRA reduced drying shrinkage and increased the time of cracking. These later crack developments typically tend to cause a decrease in crack widths, which improves the durability of bridge decks. One disadvantage of using SRAs could be its effect on strength development of low w/cm concretes. SRAs can lower the strength of these mixtures by up to 10% (at 28 days) but show no significant effect on the strength of normal strength concrete mixtures (Folliard and Berke 1997). Lura et al. (2007) studied the effect of SRA on plastic shrinkage cracking of mortars. The study found that mortars containing SRA exhibited fewer and narrower plastic shrinkage cracks than plain mortars when exposed to the same environmental conditions during the ASTM C 1579 test. It was concluded that the lower surface tension of the pore fluid in the mortars containing SRA results in less evaporation, reduced settlement, reduced capillary tension, and lower crack-inducing stresses at the topmost layer of the mortar.

1.1.2.2.5 Fiber Reinforcement

Research on fiber-reinforced concrete showed that the inclusion of fibers can significantly reduce crack size. It also tends to reduce plastic and settlement cracking (Krauss and Rogalla 1996, Qi et al. 2003, Banthia and Gupta 2006). Work by Kim and Weiss (2003) has suggested that fiber reinforcement can delay the onset and reduce the width of cracks. Experiments using a restrained ring test by Gryzbowski and Shah (1989) had shown a delay of cracking but not a discontinuity in residual strain. In later work by Shah and Weiss (2006), a decrease in strain (indicating the formation of a crack) was observed before a visible crack occurred. This is likely due to the effectiveness of fibers to bridge cracks, control crack opening (i.e., width), and prevent macrocrack propagation. In addition to reducing crack width, Qi et al. (2003) and Banthia et al. (1995) found that fibers allowed multiple cracking to occur. They also concluded that as the volume of fibers increased, the width of the cracks decreased. Micro-fibers have been found to be more effective at controlling cracking than coarse fibers (Qi et al. 2003). Much work has been performed by Li and coworkers (Li and Kanda 1998, Li 2004) to study the shrinkage performance of engineered cementitious composites (ECC) (a type of fiber reinforced mortar) and the use of this material for repair applications.

<u>1.1.2.3 Effect of Concrete's Fresh and Hardened Properties</u>

1.1.2.3.1 Slump

Previous research (Dakhil and Cady 1975, Babaei and Hawkins 1987, Schmitt and Darwin 1995) has shown a clear correlation between slump and the tendency of concrete to crack at early ages. Schmitt and Darwin (1995) believe increased slump can increase the settlement of fresh concrete over reinforcing bars and result in settlement cracking due to increased flow of the concrete around reinforcement. This is shown schematically in Figure 1-9. Poorly consolidated high-slump concrete is especially prone to this type of cracking, since vibration from construction machinery and adjacent traffic lanes can cause further consolidation and settlement after finishing and result in cracking (Krauss and Rogalla 1996). In addition, Issa (1999) attributes increased cracking in higher-slump concretes to a decrease in bond strength between the reinforcing bars and concrete.

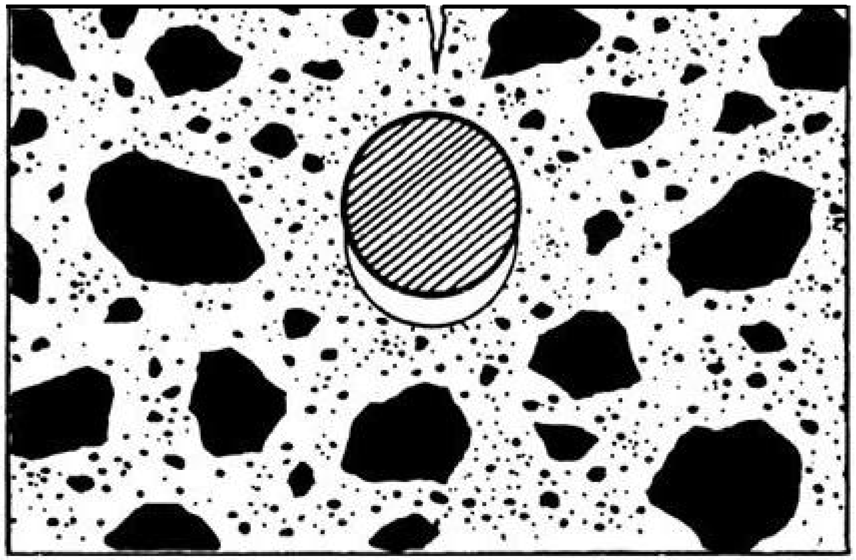

Figure 1-9: Settlement cracking due to flow of plastic concrete around rebar

Also, if increased slump is achieved by increasing paste content, concrete will be more prone to thermal and shrinkage cracks, as discussed before. Several studies have recommended reducing the slump (Babaei and Hawkins 1987, Schmitt and Darwin 1995, Issa 1999) and some have proposed values for a maximum allowable slump of 2 inches (PCA 1970), 2½ inches (Iowa DOT 1986), or 3½ to 4 inches (McLeod et al. 2009).

Schmitt and Darwin (1999) found a clear increase in crack densities with increasing slump on bridges with monolithic decks and attributed it to increasing settlement cracking. On the contrary, Cheng and Johnson (1985) observed a slight decrease in crack tendency with increasing slump in their study; however, the conclusion may be unreliable due to the little variation in slumps studied. It should also be noted that different consolidation methods were used between these two studies. Krauss and Rogalla (1996) observed zero slump concretes that tended to crack last in their study of concrete mixtures, therefore indicating a decrease in cracking with a decrease in slump.

1.1.2.3.2 Concrete Compressive Strength

An increase in the compressive strength of concrete is usually achieved by increasing cement content and reducing w/cm, which results in higher heat of hydration and higher autogenous and drying shrinkage as well as higher modulus of elasticity and lower creep. All of these conditions favor higher stress development and higher cracking risk for the concrete bridge deck. Frosch et al. (2003) showed that strengths higher than specified by structural design are not required and can exacerbate deck cracking. For example, if the structural design requires 4000 psi, specifications should impose a maximum compressive strength not far above 4000 psi.

Krauss and Rogalla (1996) related the increase in deck cracking since the 1970s to AASHTO's 1973 increase of the minimum concrete strength from 3000 psi to 4500 psi and lowering of the w/cm from 0.53 to 0.45. The work of Krauss and Rogalla (1996) also found a positive relationship between increased 7-day strength and cracking tendency of concrete. It was suggested that increases in autogenous and drying shrinkage resulted in an increase in stress development and cracking of higher strength concretes. In a study involving monolithic, overlay, and two-layer bridge decks (Schmitt and Darwin, 1999), only monolithic bridge decks had shown an increase in cracking with an increase in strength. The strength increase was related to an increase in cement content.

1.1.2.3.3 Poisson's Ratio

A reduced Poisson's ratio will reduce the shrinkage and thermal stresses to an extent in a concrete. This can be achieved through reducing compressive strengths (Krauss and Rogalla, 1996).

1.1.2.3.4 Modulus of Elasticity and Creep

Higher modulus of elasticity was found to significantly affect cracking due to increased thermal and shrinkage stresses based on Hook's law (Krauss and Rogalla 1996). Concrete's tensile strain capacity is inversely proportional to its modulus of elasticity (ACI-224R 2001). In addition, creep and stress relaxation have been found to be inversely related to the Young's modulus.

Creep is the ability of a concrete to continuously deform under a sustained stress. Relaxation is the gradual stress reduction under a sustained strain. Both creep and stress relaxation are due to the viscoelastic nature of concrete and are known to increase with the reduction of concrete's strength and elastic modulus (Mehta and Monteiro 2006).

1.1.2.3.5 Heat of Hydration

Heat of hydration depends on the amount of cement and the volume of cement paste. Higher paste contents and higher cement factors result in a higher hydration heat and temperature rise during the first 24 hours of hydration (ACI-231 2010). Other factors that affect the heat of hydration include cement type, cement fineness, batching temperature, ambient environment, and solar radiation (Riding et al. 2006). Lowering the heat of hydration lowers the thermal gradients within the concrete as well as the overall thermal contraction of concrete after setting. These reduce the risk of thermal cracking. The heat of hydration can be also reduced by proper use of supplementary cementitious materials (e.g., fly ash, blast furnace slag) as a partial replacement of Portland cement (Mindess et al. 2003).

1.1.2.3.6 Coefficient of Thermal Expansion

The coefficient of thermal expansion of concrete determines the amount of thermal strain concrete must accommodate when it is subjected to a temperature change. Most importantly, the thermal contraction of concrete after setting is directly related to its coefficient of thermal expansion. High coefficients make the concrete more susceptible to cracking, since a higher tensile strain must be accommodated. A low coefficient of thermal expansion can be achieved by increasing aggregate content and by using aggregates with lower thermal expansion coefficients. The thermal expansion coefficient of concrete is known to be significantly dependent on its moisture content (Mindess et al. 2003).

1.1.2.3.7 Concrete Thermal Conductivity

A concrete that is constructed with lower thermal conductivity (or thermal diffusivity) is expected to have greater temperature gradients throughout and therefore be more susceptible to cracking. This is because heat would flow more slowly in such concrete. Thermal conductivity of concrete is positively related to its aggregate content and moisture content, and negatively related to its porosity (Mindess et al. 2003).

1.1.3 Effect of Construction Methods on Early Age Cracking of Concrete

Research on the effect of construction methods on cracking of concrete bridge decks was conducted in several ways: use of a questionnaire, review of historical data of construction records, and through field observations or bridge instrumentation. The questionnaires assessed the causes of cracking from the viewpoint of transportation agencies, including the site conditions and construction procedures. Historical data provided information on the weather and construction procedures recorded on the day of concrete deck placement. Instrumentation and field observations gave insight on cracking tendency depending on the length of placement, placement sequence, and curing procedures.

1.1.3.1 Site Ambient Conditions

1.1.3.1.1 Air Temperature

The average air temperature at the time of placement affects concrete cracking in two different ways. Low ambient temperatures often result in a higher temperature difference between fresh concrete and ambient. As such, higher thermal stresses develop as concrete starts to cool to ambient temperature after it sets. On the other hand, high air temperatures, together with low humidity and high wind speeds, result in high evaporation rates from concrete and can increase the risk of plastic shrinkage cracking. Previous research (Schmitt and Darwin 1995) observed that air temperature affects concrete deck cracking differently depending on the type of construction. It was observed that as temperature decreased in colder construction months, the cracking increased in full-depth deck construction on continuous steel girders. For deck overlays in warmer months, on the other hand, the cracking increased with increasing temperature. A trend was found to suggest that a higher maximum daily temperature on the day of concrete placement causes an increase in cracking.

The daily temperature range also has an impact on early-age cracking. Literature (Schmitt and Darwin 1995, Krauss and Rogalla 1996) suggests that a higher temperature range results in a higher incidence of cracking. The time of placement may also have an effect on this. Bridges with evening pours tended to have less cracking, and late morning to early afternoon pours tended to be the most likely to crack. This is thought be caused by the coincidence of the concrete peak hydration temperature with the hottest time of the day (mid afternoon) when concrete is poured around noon (Krauss and Rogalla 1996). In general, placements at too high or too very low temperatures increase cracking and are not recommended (Schmitt and Darwin 1995, Krauss and Rogalla 1996). The following values for allowable ambient temperature have been proposed:

- Minimum ambient temperature: 45°F (7.2°C) (Cheng and Johnson 1985)
- Minimum and maximum ambient temperature: 40°F and 90°F (4°C and 32°C) (French et al. 1999)

In addition to ambient temperatures, the following concrete temperatures have been suggested to reduce the risk of early-age cracking:

- Maximum concrete temperature at the time of placement: 80°F (27°C) (Krauss and Rogalla 1996)
- Concrete temperature of at least 10°F to 20°F (5°C to 10°C) cooler than ambient temperature (Krauss and Rogalla 1996)
- Concrete temperature at placement 55°F to 70°F (75°F with engineer's approval) (McLeod et al. 2009)
- Girder temperature of 55°F to 75°F (12°C to 24°C) should be maintained in cold weather (Babaei and Purvis 1995)

1.1.3.1.2 Ambient Relative Humidity

Cheng and Johnson (1985) observed that low values of ambient relative humidity tended to increase the early age cracking. This is due to increased evaporation rate from the surface of plastic concrete, which can increase the risk of plastic shrinkage cracking. It is good to note that the effectiveness of moist curing to prevent drying of the concrete surface can have a large impact on mitigation of cracking.

1.1.3.1.3 Average Wind Speed and Evaporation Rate of Bleed Water

Plastic shrinkage cracking is known to be directly related to the evaporation rate of bleed water from the surface of fresh concrete (Wittman 1976, Cohen et al. 1990, Radocea 1994). Evaporation rate is a function of ambient relative humidity, concrete temperature, and wind speed and can be estimated based on the nomograph of ACI 308R-01 (TRB E-C107 2006), as shown in Figure 1-10.

Krauss and Rogalla (1996) recommended that special consideration be taken when evaporation rates exceed 0.2 lb/ft^2hr for normal concrete and 0.1 lb/ft^2hr for concrete with low w/cm. These are equal to the approximate bleeding rate of concrete, so the purpose is to ensure that the rate of evaporation remains less than the rate of bleeding so the surface of concrete never dries (ACI 308R 2001). Examples of these special considerations include installing wind breaks and fogging to reduce evaporation rates. Kochanski (1990) recommended limiting the evaporation rate to 0.25 lb/ft^2hr. Recent experience with high-performance bridge deck overlays containing silica fume show that bleeding rates are sharply reduced and as such, the maximum allowable evaporation rates should not be more than 0.05 lb/ft^2hr (Virginia DOT 1997).

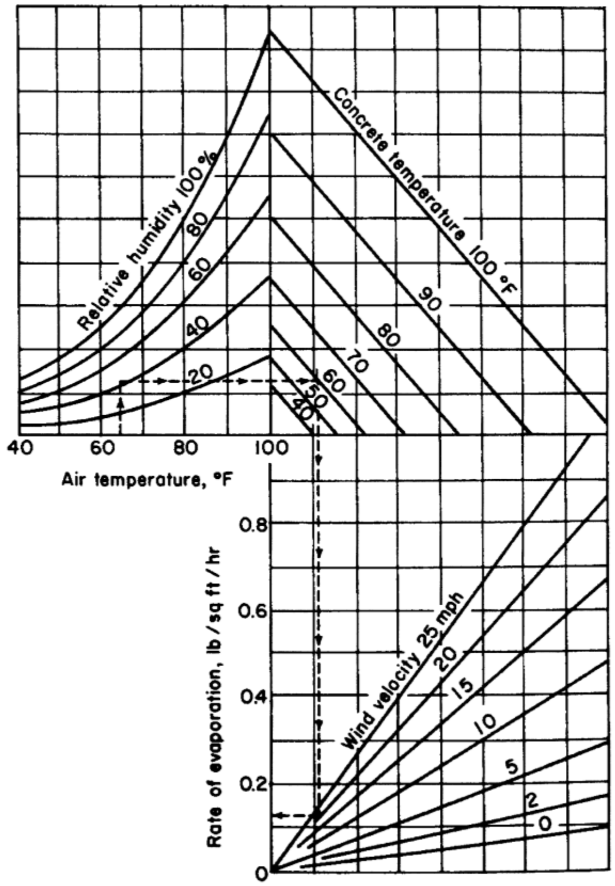

Figure 1-10: Nomograph to estimate the maximum potential rate of evaporation from concrete during curing (ACI 308R 2001)

1.1.3.2 Construction Procedures

1.1.3.2.1 Sequence and Length of Placement

Based on theoretical analysis to find the maximum curvatures and deflections with respect to various sequences of concrete deck pours, Issa (1999) noted that the sequence of pour has a considerable effect on the cracking risk of concrete. It is beneficial to start the sequence of pour in the positive moment regions and then move to the negative moment regions. The length of placement was found to have an effect for some bridge decks. An increase in the length of placement tended to increase the incidence of cracking for thinner placements where thicker decks such as monolithic bridge decks were less affected. This was mainly attributed to a delay in curing, since it took a longer time to finish the pour and consolidate the concrete (Schmitt and Darwin 1995). Ramey et al. (1997) suggested the following pouring procedure:

- Place concrete deck at one time when possible.
- Place simple span bridges, one span per placement. If the span is too long for one placement, divide the deck longitudinally and place each strip at one time. If this cannot be done, place the center of the span first and then place other portions.
- If multiple placements should be made on continuous beams, place middle spans first and observe a 72-hour delay between placements. Use bonding agents to enhance bond at joints.

1.1.3.2.2 Consolidation and Finishing

Issa (1999) found that insufficient vibration of concrete, together with insufficient cover thickness over top reinforcement, can increase plastic/settlement cracking (see Figure 9). This is especially significant for concretes with high water content and high slump. Vibration from construction machinery and adjacent traffic lanes can cause further settlement. Krauss and Rogalla (1996) found that under-vibration of concrete tended to cause an increase in cracking where over-vibration had little effect. They also studied effects of construction loads, tightness of reinforcement ties, vibration from traffic, and revolutions in the concrete truck. These parameters were found to have only minor effects on early-age cracking. McLeod et al. (2009) proposed detailed vibration practices to produce low cracking concrete in Kansas.

Literature (Krauss and Rogalla 1996) suggests that delayed finishing causes an increase in cracking. Double-floated finishing produced concrete with less cracks than the standard float method.

1.1.3.2.3 Curing

Several transportation agencies surveyed considered curing to have one of the highest impacts on the occurrence of early-age cracking on concrete bridge decks. Proper curing reduces cracking caused by plastic shrinkage in fresh concrete. Delayed curing tends to increase cracking risk. Concretes with a high cement factor and low w/cm are affected more by a delay in curing due to less bleeding for low w/cm. Moist curing and chemical evaporation retarder films prove to help decrease the number of cracks formed during curing. Curing should begin immediately after finishing (Bentz and Jensen 2004, Krauss and Rogalla 1996, Schmitt and Darwin 1995). When wet burlaps are used, it has been recommended to apply the first layer of pre-soaked burlap 10 minutes after strike-off and apply a second layer within 5 minutes (McLeod et al. 2009). Moist curing should continue for a minimum of 7 days (Frosch et al. 2003) but 14 days is more preferred (NYSDOT 1995). It is recommended to apply opaque curing compound to the surface of concrete after 14 days of wet curing (McLeod et al. 2009). Krauss and Rogalla (1996) recommended the following procedure for curing:

- Use of fog nozzle water spray in hot weather to cool the concrete and to cool the steel and forms immediately prior to placement. Ponding of water on the forms or plastic concrete should not be allowed.
- Use of wind breaks and enclosures when the evaporation rates exceed 0.2 lb/ft^2hr for normal concrete or 0.1 lb/ft^2hr for low w/cm concretes susceptible to plastic cracking. The ACI 308 nomograph should be used to estimate evaporation rates.
- Application of water mist or monomolecular film immediately after strike-off or early finishing.
- Application of white-pigmented curing compound as soon as bleed water diminishes.
- Application of pre-wetted burlap as soon as concrete resists indentation. The burlap must be kept continuously wet by continuous sprinkling or by covering the burlap with plastic sheeting and periodic sprinkling.
- Continuation of wet curing for a minimum of 7 days, preferably 14 days. Curing should be extended in cold weather until the concrete has gained adequate strength.

1.1.4 Effect of Structural Design Factors on Early Age Cracking

Structural design factors that have been researched for their effect on bridge deck cracking can be categorized best into bridge deck design, girder and span conditions, and loading. Research on the effect of structural design has been conducted by field observations as well as theoretical analysis using finite element or similar analysis programs.

1.1.4.1 Bridge Deck Design

1.1.4.1.1 Structure Type

Schmitt and Darwin (1995) found no increase in cracking tendency with respect to four different structure types, including steel beam composite continuous, steel welded plate girder composite continuous, steel welded plate girder composite continuous and haunched, and non-composite bridge deck structures. Studies on span and girders are discussed later in this chapter.

1.1.4.1.2 Deck Type

Several studies (PCA 1970, Cheng and Johnson 1985, Krauss and Rogalla 1996, Frosch et al. 2002) have found that decks on steel girders tend to crack more when compared to decks on concrete girders. It is believed that since concrete girders conduct heat more slowly than steel girders (i.e., resulting in lower temperature gradients inside newly placed concrete), thermal stresses in the deck of bridges with concrete girders are lower than for steel girder bridges. On the other hand, Schmitt and Darwin (1995) studied 40 bridges throughout the State of Kansas and compared crack densities for different variables that could affect cracking on bridge decks. Their study did not find a strong correlation to indicate that the type of deck had any effect on the occurrence of cracking.

1.1.4.1.3 Deck Thickness

Some studies have found a decrease in cracking with increased deck thickness (Poppe 1981, Kochanski 1990, Ramey et al. 1997, French et al. 1999). Krauss and Rogalla (1996) found this effect to be inconsistent in their analytical study, possibly due to non-uniform shrinkage and thermal stresses in the concrete. A study by Saadeghavaziri and Hadidi (2005) utilized 2D and 3D finite elements to analyze stresses in bridge decks. They found that increasing deck thickness decreased the stresses in the deck, with the exception of those with a fixed-fixed boundary condition. This is due to a decrease in the degree of restraint for thicker decks, which directly impacts shrinkage stresses and cracking (Moon et al. 2006).

1.1.4.1.4 Top Cover

A mid-range top cover was found to exhibit the least amount of cracking tendency when top cover depths of 2 to 3 inches (51-76 mm) on monolithic bridge decks were researched (Schmitt and Darwin, 1995). Krauss and Rogalla (1996) also suggested at least a 2-inch (50 mm) top cover to avoid settlement cracking. A cover that is too low increases the chances of settlement cracking, where a high cover thickness reduces the effectiveness of the reinforcing bars to distribute stresses and to reduce the crack widths. AASHTO LRFD Bridge Design Specifications, Section 5.7.3.4 states that the top cover shall not be less than 2.5 inches.

1.1.4.1.5 Reinforcement

A study for Wisconsin DOT suggested that increasing the reinforcement bar size would increase the cracking on bridge decks (Kochanski, 1990) (also Dakhil and Cady 1975). Other studies (Babaei and Hawkins 1987, Schmitt and Darwin 1995) have also observed the same behavior and have recommended limiting the deck bar size. Kochanski et al. (1990) as well as Ramey et al. (1997) recommend the use of a maximum bar size of No. 5.

Increased spacing between transverse reinforcement was also shown to increase cracking, but this could also be due to larger bars used with the increased spacing. Some literature recommends using smaller bars at smaller spacing (French et al. 1999). Frosch et al. (2003) discussed that the current AASHTO code requirements for shrinkage and temperature reinforcement do not place sufficient limits on bar spacing and suggested a maximum bar spacing of 6 inches. This agrees with recommendations of Krauss and Rogalla (1996). To prevent transverse cracking of bridge deck, NYSDOT (1995) recommends placing the longitudinal bars on top of the transverse bars. To prevent shrinkage cracking in a repair patch next to a deck's dam replacement, transverse bars should be placed on top of longitudinal bars (when possible) to reduce the risk of longitudinal settlement cracking.

Other reinforcement-related issues arising in concrete are the type, alignment, and quantity of rebar. Transportation agencies have noted that epoxy-coated bars increased cracking (Krauss and Rogalla, 1996). This was supported by earlier research findings (Meyers 1982) suggesting that the bond strength between the concrete and the epoxy-coated bars is less than the bond with uncoated bars. Krauss and Rogalla (1996) found top and bottom reinforcement bars that are vertically aligned tended to increase cracking and resulted in full-depth cracks. However, Saadedghvaziri and Hadidi (2005) did not find an increase in cracking potential due to the arrangement of reinforcement bars. The quantity of reinforcement used should be greater than specified in the AASHTO design manual (Krauss and Rogalla 1996). AASHTO LRFD Bridge Design Specifications, Section 5.10.3 states that the clear distance between parallel reinforcing bars shall not be less than 1.5 times the nominal diameter of the bars, 1.5 times the MSA, or 1.5 inches.

1.1.4.1.6 Other Deck Design Considerations

Other factors of the deck design have also been assessed in the literature, including stud spacing, use of post-tension design, form type, and skew. Stud spacing was not observed to cause an increase in cracking; however, finite element simulations showed a 20% higher-than-average stress concentration at stud locations (Krauss and Rogalla, 1996). Krauss and Rogalla (1996) have found that girder restraint and stud type cause significant cracking; however, they do not provide any suggestion on how to reduce girder restraint through changes in stud configuration and properties. French et al. (1999) have recommended fewer studs with smaller rows and lengths, but they do not provide specific guidelines.

The AASHTO design specification for post-tensioned deck design was found to increase cracking when additional reinforcement is not used in the tensile zones. Krauss and Rogalla (1996) explained that the design procedure is adequate to address longitudinal movement at the supports but it does not address tensile stresses from shrinkage and daily temperature changes. In the literature review performed by Krauss and Rogolla (1996) prior to their analytical study, they found that some earlier work regarding stay-in-place (SIP) forms showed a decrease in transverse cracking. Other research, however, found that SIP forms caused an increase in tensile stresses due to non-uniform shrinkage and therefore more cracking occurred at the upper deck surface (Schmitt and Darwin, 1995). Some research found an increase in cracking when the skew was greater than 30 degrees (Schmitt and Darwin, 1995). AASHTO LRFD Bridge Design Specifications do not place any absolute limit on the spacing of the reinforcing bars.

1.1.4.2 Girders and Spans

1.1.4.2.1 End Condition

In general, fixed-end girders increased cracking in the end sections of bridge decks compared to pin-ended girders. Fixed-end girders provide a higher degree of restraint, and therefore increase the potential for cracking (Schmitt and Darwin 1995). Saadeghvaziri and Hadidi (2005) studied four end condition types in a 3D finite element analysis of bridge decks. The four conditions were: pin-roller, pin-pin, fixed-roller, and fixed-fixed. They found that as the system becomes stiffer, the distance between cracks decreases and the amount of strain needed for a crack to develop also decreases. French et al. (1999) observed a better performance (less cracking) for simply-supported pre-stressed girder bridges compared with continuous steel girder bridges due to reduced end restraint in the former.

1.1.4.2.2 Girder Type

Transportation agencies reported that steel girders increased cracking due to varying stiffness and thermal properties between steel and concrete (Krauss and Rogalla, 1996). This was supported by analytical studies. This is thought to be caused by differences in thermal conductivity of the steel compared to the concrete. Krauss and Rogalla (1996) found that cast-in-place concrete girders and young pre-stressed girders have the best performance, while deep steel beams have performed worse. Larger girders typically caused more cracking (higher degree of restraint) as well as girders placed at longer distances (Schmitt and Darwin, 1995). Saadeghvaziri and Hadidi (2005) found this to be true in only a fixed-fixed end condition. They suggested that increasing the girder spacing may reduce the tendency for deck cracking. It is suggested that girder size and spacing be minimized (Schmitt and Darwin 1995). External restraint commonly leads to cracking; however, some measures can be taken to control where the cracks initiate. Contraction or control joints can be used to avoid uncontrolled cracking (Bentz and Jensen 2004). Saadeghvaziri and Hadidi (2002) showed that it is the relative stiffness of the deck with respect to the girder stiffness that is more critical than the type of girder.

1.1.4.3 Loading

Heavy truck traffic was found to increase the length of cracks in bridge decks, but a clear trend in data as to the significance of this loading to cause cracking was not established (Krauss and Rogalla 1996). Overall, tensile stresses caused by mechanical loading of the bridge were found to be far smaller than the stresses generated by restrained shrinkage and thermal contraction (Schmitt and Darwin 1995, Hadidi and Saadeghvaziri 2005, Frosch et al. 2003).

1.1.5 Other Methods for Reducing Early Age Cracking

In addition to the studies summarized earlier, a number of computer programs have been developed to simulate cement hydration, evolution of mechanical properties, and durability performance of concrete in aggressive environments. As it relates to the objectives of this study, a number of computer programs exist that are capable (with different degrees of accuracy and reliability) of assessing the risk of early-age thermal cracking of concrete members. The models predict concrete temperatures, thermal strains, and resulting tensile stresses due to restraining these volume changes. By comparing the magnitude of stress to the tensile strength of concrete at each age, the programs assess the risk of thermal cracking. These models are briefly described below.

1.1.5.1 HIPERPAV III

HIPERPAV III (Ruiz et al. 2005, HIPERPAV III 2011) is a simulation program for determination of the early age thermal cracking behavior of Portland cement concrete (PCC) pavements during the first 72 hours after construction. The program uses four groups of input information: pavement design, materials/mix design, construction, and environmental conditions. Specific details about these four input categories are used to generate stress and strength development models for the concrete placement under consideration. Two of the outputs of this early-age concrete analysis are critical stress and strength development during the first 3 days following the PCC pavement construction.

1.1.5.2 ConcreteWorks

ConcreteWorks (2005) is designed to be a user-friendly concrete mixture proportioning, thermal analysis, and chloride diffusion service life software package. The software contains design modules for several mass concrete shapes, bridge deck types, precast concrete beams, and concrete pavements. Input groups consist of: general inputs, shape inputs, member dimensions, mixture proportions, material properties, mechanical properties, construction inputs, environmental inputs, and corrosion inputs. Temperature prediction analysis is available for both bridge decks and pavements.

1.1.5.3 eVCCTL

The Virtual Cement and Concrete Testing Laboratory (VCCTL) software has been developed by the National Institute of Standards and Technology (NIST). The educational version (eVCCTL) is currently available, which provides a virtual testing laboratory environment that can be used by concrete scientists, engineers, and technologists to explore the properties of cement paste and concrete materials (Bullard 2010). With this software the user can:

- Create virtual materials, using carefully characterized cement powders, supplementary cementitious materials, fillers, and aggregates;
- Simulate the curing of these materials under a wide range of conditions; and
- Calculate their thermal, mechanical, and transport properties as a function of their processing.

1.1.5.4 Femmasse HEAT

The description of this program is to design measures to prevent early-age cracking of concrete (Femmasse 2011). HEAT makes it possible to analyze the effect of the following concrete placement conditions and measures intended to prevent early-age thermal cracking: composition of the concrete, type of formwork, time of formwork stripping, artificial cooling, heating wires, location of joints, and insulation. Using its extensive materials database, HEAT can analyze the effect of different types of concrete. The program includes a large number of material parameters, including adiabatic heat development, maturity and degree of hydration, heat and moisture diffusion coefficients, thermal and hygral dilation, strength development, visco-elastic properties, and fracture properties.

1.1.5.5 DuCOM

DuCOM is a finite element-based computational program used to evaluate various durability aspects of concrete (DuCOM 2011). DuCOM stands for Durability Models of COncrete. The current version (2.21) traces the development of concrete hardening (hydration), structure formation, and several associated phenomena from initial casting of concrete and extending for a period of several months or even years afterwards. As such, this tool can be utilized to study the effects of ingredient materials and environmental conditions as well as the size and shape of the structure on the durability of concrete. The term durability considered here takes into account both early-age concrete problems as well as mature concrete exposed to aggressive environments. This tool can be used to analytically trace the evolution of microstructure, strength and temperature over time for any arbitrary initial and boundary conditions. Since the main simulation program is based upon finite-element methods, it can be applied to the analysis of real-life concrete structures of any shape, size, or configuration. Further, dynamic coupling of several phenomena ensures that the effects of changing environmental conditions are easily integrated into the overall simulation scheme.

1.2.0 RESULTS OF SURVEY OF TRANSPORTATION AGENCIES

A questionnaire on bridge deck dam cracking was sent to the DOT bridge engineers in 50 states, 11 PennDOT districts and the cities of Harrisburg, Pittsburgh, and Philadelphia. The questionnaire can be found at the end of this section. At the time of finalizing this report, 21 states and 4 PennDOT districts have replied to the questionnaire. Of the 25 responses, 3 states and 1 PennDOT district have indicated that they do not replace expansion joints/dams on concrete bridge decks without replacing the entire deck. A number of key questions on the subject are presented in this section along with statistics of responses.

1.2.1 Frequency of Replacing Expansion Dams

The frequency with which the agencies have been replacing expansion dams on existing bridges is shown in Figure 1-11.

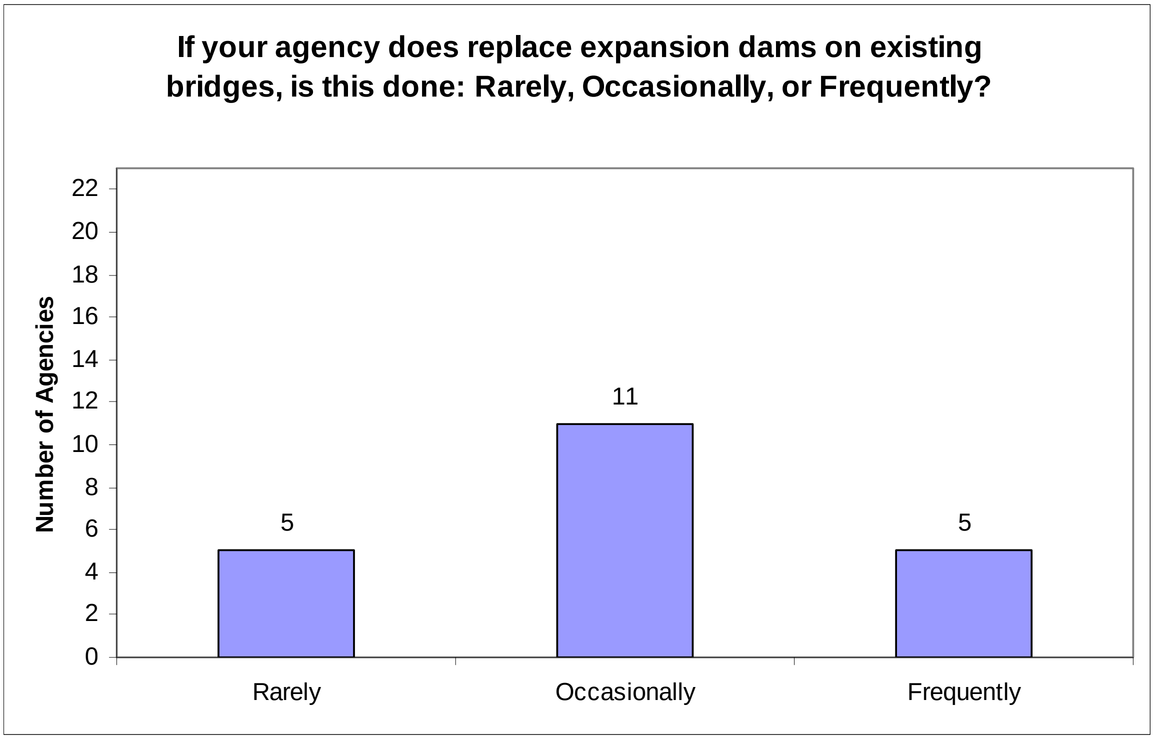

Figure 1-11: The frequency of replacing expansion dams on existing bridges

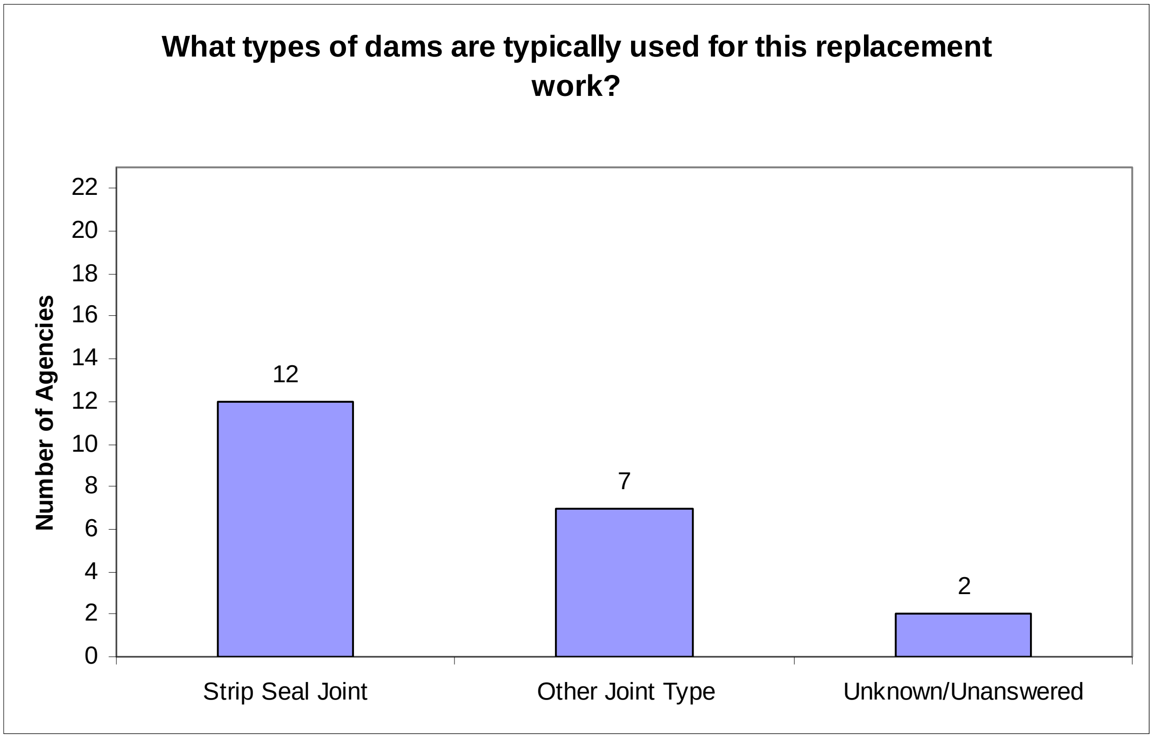

Figure 1-12: Types of joints used in the replacement work

1.2.2 Typical Types of Dams for Replacement

The number of agencies using different types of dams is shown in Figure 1-12. For the agencies that reported using strip seal joints plus other types, the strip seal joint was recorded in order to differentiate the agencies that do not use this type of joint. Other types of joints used by the agencies include finger joints, sliding plates, armored joints, compression seals, membrane sealants, armor-less joints with header, modular expansion joints, steel angles, polymer-modified concrete, and elastomeric concrete.

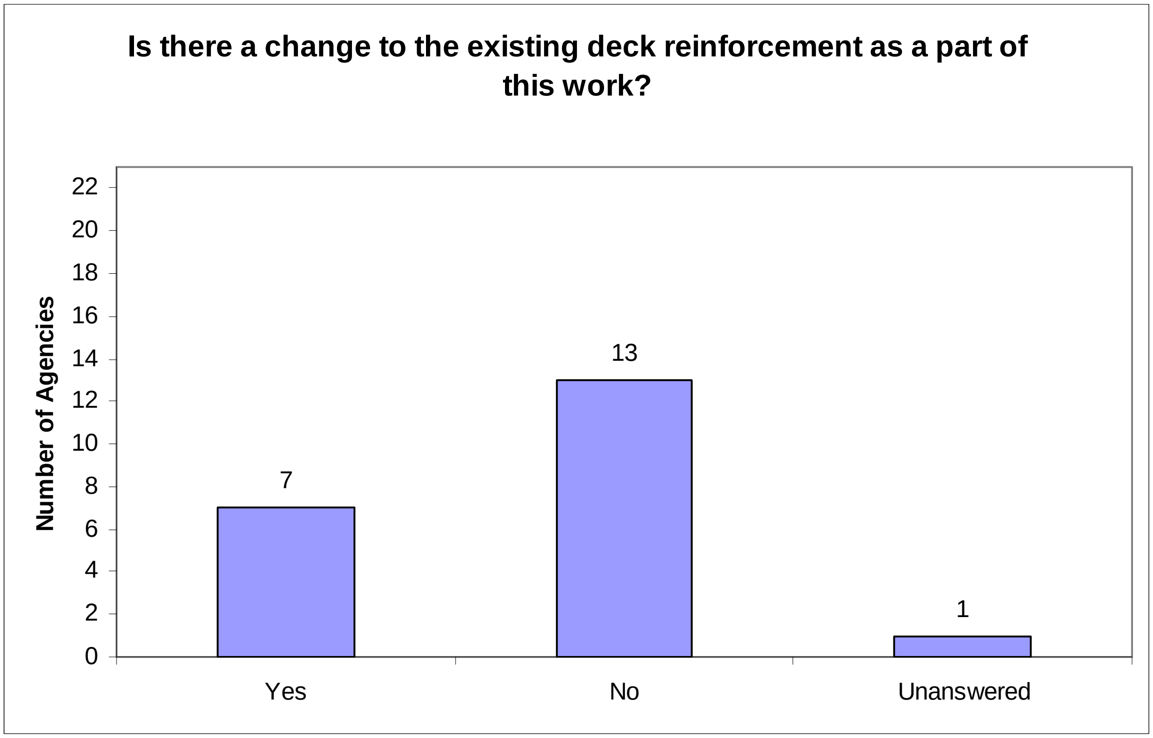

Figure 1-13: Change/No change to the existing deck reinforcement

1.2.3 Change in the Existing Deck Reinforcement

The number of agencies that change the existing deck reinforcement as part of the work versus the ones that do not change the existing deck reinforcement is shown in Figure 1-13. Kansas DOT changes the existing deck reinforcement on a case-by-case basis. Typically, the existing rebar is reused if it is in good shape and replaced if it is deteriorated. Additional rebar is added to the new concrete block-out to tie the repair patch to the existing concrete. Adequate rebar development lengths or splice lengths are provided. Minnesota DOT keeps the longitudinal rebar in place, and calls for their cleaning and straightening. New transverse bars (parallel to the joint) are added in the repair section. They have provided detail drawings for this work as well.

For replacements involving new strip seal joints, supplementary reinforcing may be provided by doweling into existing portions of deck and approach slab during work for Nevada DOT. Standard details or guidelines haven't been developed for this work. New Mexico DOT generally thickens the concrete deck slab and adds additional reinforcing. In other cases, they pour a solid concrete diaphragm on both sides of the expansion joint.

PennDOT District 11 indicated that its goal on rehabilitation projects is to look at the existing plans and try to get the bridge as close to the current standard as possible by adding reinforcement bars. Tennessee DOT removes transverse deck reinforcement and replaces it within the concrete removal area. They have also provided a link to the replacement details. Utah DOT replaces bars that lie parallel to the joint and are exposed by concrete removal operations. New expansion joints are anchored with 5/8-inch diameter bars attached to ½-inch plate spaced at 1 foot. They have also provided drawings from a recent project.

1.2.4 Experiencing Cracking of the New Deck

Figure 1-14 shows the number of agencies that have experienced cracking of the new deck concrete placed during the expansion dam replacement versus the ones that have not experienced cracking in concrete. Maryland State Highway Administration (MSHA) indicated that it has had some cracking in the joint repairs, but these would be infrequent and MSHA has not considered it much of a problem. The agency typically attributes these problems to a lack of quality control during mixing of the material (e.g., improper w/cm) or to improper curing of the material (e.g., temperature too high or too low for the particular concrete mixtures used). With regard to the cracking addressed in the survey, Nevada DOT (NDOT) states that it has been most prevalent in the concrete pour-backs next to the expansion joint header for new construction. NDOT has also provided a specification for the concrete typically used for these pour-backs.

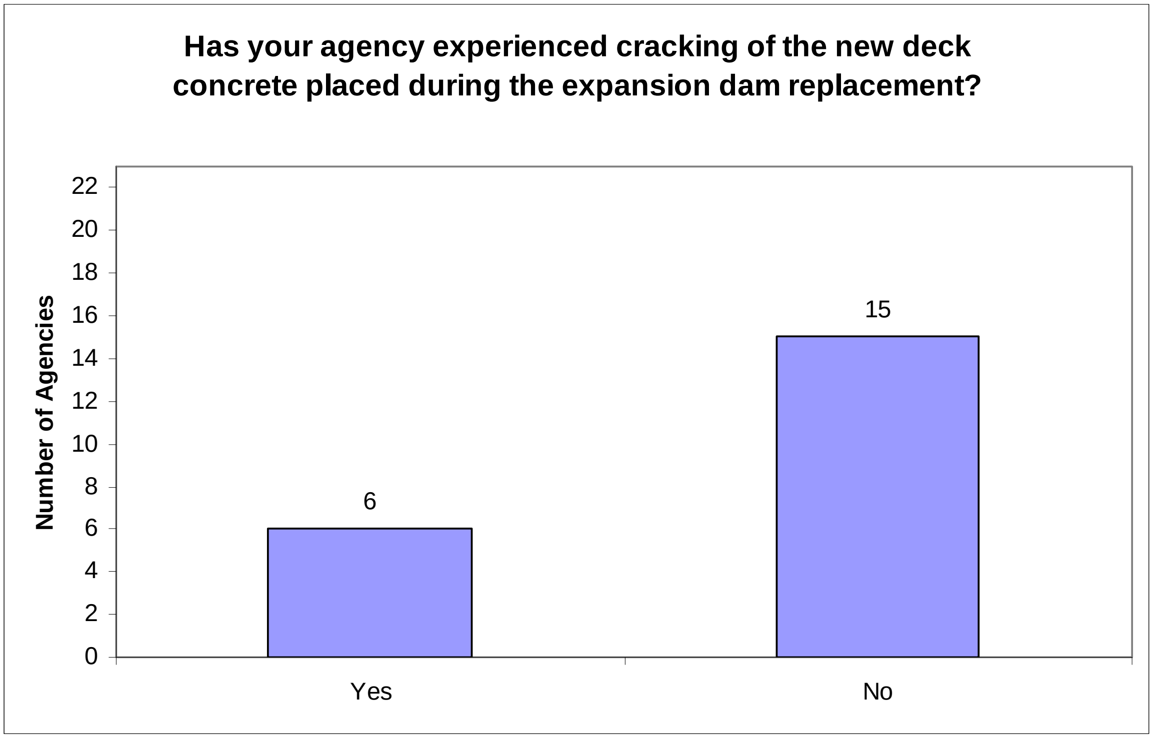

Figure 1-14: Cracking/No cracking in the new deck concrete placed during the expansion dam replacement

1.2.5 Elimination of Cracking

Of the 6 agencies that reported cracking in concrete, one agency indicated that it has successfully eliminated the cracking problem. New York DOT (NYSDOT) has stated that it eliminated cracking by having a separate header pour after deck repairs are made. NYSDOT has provided drawings of the joint system. The Tennessee DOT engineer mentioned that elimination of cracking could probably be accomplished with a wet cure on the header (dam), but he also mentioned that with the lack of proper project inspection/supervision, it is hard to get contractors to do this. On eliminating the cracking problem, Wyoming DOT recommends the use of Portland cement concrete instead of other materials, and to make sure the new concrete is reinforced and is not placed in thin applications. Nevada DOT has seen diminished incidence of cracking (but not complete elimination) since it implemented the use of high performance concrete and wet curing of bridge decks and approach slabs.

The following exhibit shows the questionnaire that was distributed among transportation agencies.

Longitudinal Cracking in Concrete at Bridge Deck Dams on Structural Rehabilitation Projects Questionnaire

Introduction

The Pennsylvania Department of Transportation (PennDOT) has asked a Penn State University research team to conduct a survey as part of a research project entitled, "Longitudinal Cracking in Concrete at Bridge Deck Dams on Structure Rehabilitation Projects". The results of the survey will assist PennDOT in identifying other agencies with similar issues, as well as experiences in mitigating concrete cracking associated with bridge expansion dam replacement.

As a member of the Penn State research team Quality Engineering Solutions (QES) is requesting your assistance in the form of information regarding the replacement of bridge expansion dams during rehabilitation work. We are specifically interested in obtaining information from other agencies about their policies and experiences with bridge expansion dam replacement. Specifically, we are seeking experience related to concrete cracking associated with bridge expansion dam replacements, but more general information is also requested in the attached brief questionnaire.

If you are not the appropriate contact person, please let us know who we should contact.

We greatly appreciate your assistance in providing this information. We would be grateful if you send the completed questionnaire back within 2 to 4 weeks of its delivery.

Please send the completed questionnaire via e-mail at:
sjahangirnejad@QESpavements.com.

Questions

1. Respondent information:
   Contact Name/Title: /
   Agency/District, Division, etc.: /
   Phone No./Email: /
2. Does your agency replace expansion joints/dams on concrete bridge decks without replacing the entire deck? ☐ Yes ☐ No
   If no, please provide your contact information, and thank you for your assistance.
3. If your agency does replace expansion dams on existing bridges, is this done:
   ☐ Rarely? ☐ Occasionally? ☐ Frequently?

4. Is work of this nature performed on structures with:
   ☐ Concrete beams? ☐ Steel beams? ☐ Other structure types?
5. What types of dams are typically used for this replacement work?
6. Is there a change to the existing deck reinforcement as a part of this work?
   ☐ Yes ☐ No
   If yes, please provide information showing the revision.
7. Has your agency experienced cracking of the new deck concrete placed during the expansion dam replacement process? ☐ Yes ☐ No
   If yes, is cracking ☐ longitudinal or ☐ transverse?
   If you have experienced cracking, please provide an electronic link to specifications, or information which will enable us to find the description of the type of concrete being used in this repair.
8. Please provide your design and construction guidelines (electronic or other media) for this type of work.
9. Has your agency conducted any research on this topic? ☐ Yes ☐ No
   If yes, please provide information indicating how that work can be acquired.
10. Has your agency successfully eliminated this previous problem? ☐ Yes ☐ No
    If yes, please provide information explaining how this was accomplished.
11. Please provide contact information for the most appropriate person within your organization to discuss this work in greater detail.
    Contact Name/Title: /
    Phone No./Email: /

1.3.0 SUMMARY AND CONCLUSIONS

Tables 1-2, 1-3, and 1-4 summarizes the effect of concrete material properties, construction practices, and structural design factors on the early-age cracking tendency of restrained concrete elements in a bridge deck. Only the parameters that are believed to influence cracking in repair sections adjacent to bridge dam replacements have been included.

Table 1-2: Effect of concrete's proportions and material properties on the risk of early-age cracking

Material Property	Effect on Cracking	Material Property	Effect on Cracking
Water to Cement Ratio Too High or Too Low	↑	Slump	↑
Cement Content	↑↑	Concrete Compressive Strength	↑
Water Content	↑↑	Modulus of Elasticity	↑↑
Aggregate Content	↓↓	Creep	↓
Air Content	↔	Heat of Hydration	↑
Cement Type	↑↓	Concrete Coefficient of Thermal Expansion	↑
Chemical Admixtures	↑↓	Concrete Thermal Diffusivity	↓
Mineral Admixtures	↑↓		
Fiber Reinforcement	↓		

Table 1-3: Effect of concrete construction practices on the risk of early-age cracking

Construction Method	Effect on Cracking	Construction Method	Effect on Cracking
Low or High Air Temperature	↑	Inadequate Curing	↑↑
Low Ambient Relative Humidity	↑	Insufficient Consolidation	↑
High Wind Speed	↑		

Table 1-4: Effect of structural design factors on the risk of early-age cracking

Structural Design Factor	Effect on Cracking	Structural Design Factor	Effect on Cracking
Lower Deck Thickness	↑	Larger Reinforcement Bar Size and Spacing	↑
Low Cover Thickness	↑↑		

Based on the review of available literature, the following general recommendations can be made to reduce the risk of early-age cracking in concrete repair sections next to replaced deck dams:

- The risk of plastic shrinkage cracking must be reduced. This can be achieved by:
    a) Proper moist curing for at least 7 days so the surface of concrete never dries; moist curing should start as soon as it is practically possible after strike off.
    b) Monitoring site ambient condition (temperature, humidity, wind speed) at the time of pour; and if needed, adopting strategies to limit evaporation rate
    c) Limiting concrete slump

- Drying shrinkage of concrete must be reduced by:
    a) Optimizing/maximizing aggregate content (i.e., reducing cement content)
    b) Avoid using too high or too low w/cm
    c) If needed, use of shrinkage reducing admixtures

- The risk of autogenous shrinkage and thermal cracking must be reduced by:
    a) Limiting the allowable cement content
    b) When appropriate, use of supplementary cementing materials
    c) Monitoring heat of hydration is recommended

- Use of excessively strong concretes should be avoided. PennDOT may consider imposing a maximum allowable compressive strength above the structural design requirements. This is also to avoid high elastic modulus, which is known to increase the risk of early-age cracking.

- Better choice of reinforcing bars can help reduce the potential for early-age cracking:
    a) Shrinkage and temperature reinforcement must at least satisfy AASHTO code design requirements. Some studies have suggested that current code requirements could be insufficient.
    b) Use of large bar sizes and large spacing between reinforcing bars must be avoided. Future special provisions may need to limit the maximum bar size and spacing.
    c) When possible, use transverse reinforcement on top of longitudinal reinforcement to reduce the risk of longitudinal cracking due to plastic shrinkage, drying shrinkage, and mechanical loading.

A number of state transportation agencies interviewed stated the significance of proper moist curing to prevent early-age cracking. The results of this literature review will assist in better evaluating the current PennDOT specifications as well as design and construction documents associated with past projects and current bridge deck dam replacement projects. In the research project's Task 2 (chapter 2), information from past and active projects is compared with recommendations from the literature as well as findings from the survey of other transportation agencies to determine areas that improvements can be made.

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CHAPTER 2 (TASK 2)

Review of PennDOT Specifications as well as Past and Present Bridge Deck Dam Rehabilitation Projects to Evaluate Causes of Early-Age Cracking

2.1.0 INTRODUCTION

This chapter presents and discusses the results of a review of PennDOT specifications as well as design and construction documentation associated with a number of past and active projects. This review investigates the concrete mix design, the steel reinforcing bar design, and the construction practices implemented by contractors. Photographs and observations from site visits are also presented. The variables in design, construction, and contractor practices are listed in a matrix format.

The past PennDOT District 3-0 bridge deck dam rehabilitation projects reviewed during Task 2 (this chapter) are jobs 15-7PP, 180-044, and 180-058. These projects adhere to PennDOT Specifications Publication 408. Over the years, this publication has been revised and is oftentimes revised multiple times in the same year. Since the bridge deck dam rehabilitations occurred in different areas during different years, the rehabilitations were therefore performed according to different versions of Publication 408. Project 15-7PP is specified to *PennDOT Specifications Publication 408/2007 Initial Edition*. Project 180-044 is specified to *PennDOT Specifications Publication 408/2003 Change Number 4*. Project 180-058 is specified to *PennDOT Specifications Publication 408/2007 Change Number 1*. As is noted during this study, there have been only minor changes with respect to the *PennDOT Specifications Publication 408* with respect to the handling of bridge deck dam rehabilitations over the last decade. The greatest change can be attributed to the specification change allowing new cement concrete mixtures, HPC and AAA-P, to be implemented during construction. However, none of the past projects evaluated by the PSU team had used the concrete mixture AAA-P or HPC mixtures. The latter is a modified AAA mixture developed by PennDOT District 3-0.

The aforementioned *PennDOT Specifications Publication 408* revisions are reviewed along with the Publication 408 that is the most recent as of the start of the present study (Publication

408/2011 Change Number 1, October 7, 2011). The objective is to review these documents, note any major changes, note any ambiguity, and seek clarifications when necessary.

Present PennDOT projects occurring in District 2-0 and District 3-0 were observed to better understand field adherence to construction specifications. A team of Penn State University (PSU) and Quality Engineering Solutions (QES) engineers observed the construction practices of District 2-0 and the contracting team of Glenn O. Hawbaker three times in May 2012 at a project located on state route 322 at section Z08 in Derry and Brown Townships in Mifflin County. This same team then traveled to Sullivan County in District 3-0 to observe the rehabilitation of bridge dams at a project along state route 87 at Section 65M over Little Loyalstock Creek.

Along with the review of past and present PennDOT specifications, as-built drawings for projects 15-7PP, 180-044, and 180-058 were also reviewed. These as-built drawings allow post-construction insight into the adequacy of the reinforcing steel design against cracking due to stresses initiated by concrete shrinkage and temperature variations as well as stresses caused by service loads. The information gathered in the review of construction specifications and as-built drawings is compared to the literature recommendations as reported in chapter 1. Tables 2-1 and 2-2 present the factors that most importantly affect the early-age cracking tendency of restrained concrete elements.

Table 2-1: Effect of material properties on the risk of early-age cracking

Material Property	Effect on Cracking	Material Property	Effect on Cracking
Water to Cement Ratio	↑↓	Slump	↑
Cement Content	↑↑	Concrete Compressive Strength	↑
Water Content	↑↑	Modulus of Elasticity	↑↑
Aggregate Content	↓↓	Creep	↓
Air Content	↔	Heat of Hydration	↑
Cement Type	↑↓	Concrete Coefficient of Thermal Expansion	↑
Chemical Admixtures	↑↓	Concrete Thermal Diffusivity	↓
Mineral Admixtures	↑↓		
Fiber Reinforcement	↓		

Table 2-2: Effect of concrete construction practices on the risk of early-age cracking

Construction Method	Effect on Cracking	Construction Method	Effect on Cracking
Low or High Air Temperature	↑	Inadequate Curing	↑↑
Low Ambient Relative Humidity	↑	Insufficient Consolidation	↑
High Wind Speed	↑		
Lower Deck Thickness	↑	Larger Reinforcement bar size	↑
Low Cover Thickness	↑↑		

2.2.0 REVIEW OF PENNDOT STRUCTUAL DESIGN AND CONSTRUCTION SPECIFICATIONS

This section reviews the construction specifications according to PennDOT Specifications Publication 408 as well as the structural design based on as-built drawings. The steel requirements calculations based on temperature and shrinkage as well as strength (i.e., to safely carry service loads) are reported in Appendix A, where the calculations are compared to ACI 318-11 and AASHTO LRFD Bridge Design Specifications.

2.2.1 Steel Reinforcing Bar Requirements

As discussed in Task 1, longitudinal cracking in repair sections near bridge deck dams is similar in nature to transverse cracking of newly constructed bridge decks. In both cases, restrained shrinkage results in tensile stress development and cracking that is predominantly in the direction perpendicular to the longest dimension of the concrete member (i.e., transverse for full bridge decks, and longitudinal for dam repair patches). This scenario may be exacerbated in dam repairs due to a higher degree of restraint and especially if rapid hardening concretes are used. In addition, plastic shrinkage, inadequate curing, and structural design deficiencies can contribute to cracking of both bridge decks and repair sections. This section of this study has, therefore, focused on steel reinforcing bar design requirements based on transverse steel reinforcement specification for the dam repair.

A wide variety of repair materials are available for rehabilitation of bridges and can be classified into three primary groups: cementitious mortars and concretes, polymer-modified cementitious concretes, and epoxy-binder concretes (Emberson and Mays 1990, Cusson and Mailvaganam 1996). Among these, a properly designed, placed, and cured conventional Portland cement concrete remains as one of the most reliable, durable, and cost-effective repair materials (Parameswaran 2004). Latex-modified and other types of polymer-modified concretes can be used to improve bonding of the repair to its substrate. PennDOT has retrofitted cracked concrete sections of bridge decks with an epoxy overlay and has since made it a common practice to specify latex- or epoxy-based overlays on all bridge dam rehabiliations. None of the past projects reviewed in District 3-0 specified the use of a latex overlay; however, upon noticing the crack propagation, such an overlay was applied. It is noted that some of the bridge deck dam rehabilitions are over diaphragms, which could contribute to both the concrete restraint as well as the strength and resistance to cracking for the newly constructed dams.

Past research has suggested that top cover for reinforcing bars should be at least 2 inches to mitigate corrosion of reinforcing steel due to penetration of deicing salts. Adequate cover thickness has the added benefit of preventing cracking induced by settlement of concrete around the top rebar layer (Schmitt and Darwin 1995, Krauss and Rogalla 1996). Research has also recommended limiting the bar size and spacing between the reinforcing bars to reduce the risk of cracking. Ramey et al. (1997) suggested the maximum reinforcing bar size of #5. Research suggests a maximum rebar spacing of 6 inches (Frosch et al. 2003), to which PennDOT adheres in the transverse direction.

The steel reinforcing bar analysis conducted by the research team is presented in Appendix A. Calculations based on both ACI and LRFD temperature and shrinkage steel requirements were conducted for comparison. Along with the equations and calculations, tables A1 through A3 provide information on the bridge dams being rehabilitated, the depth of the deck, the type of overlay applied, the length of the deck replaced, the area of bridge that was repaired, the reinforcing bar numbers, the rebar amount, the rebar cross sectional area, and the calculated steel requirements according to ACI and LRFD. These tables indicate that PennDOT bridges constructed during projects 15-7PP, 180-044, and 180-058 are adequate according to temperature and shrinkage steel requirements.

Appendix A also provides standard calculations for positive and negative bending moment reinforcing bar design as well as concrete cracking control calculations. These equations and calculations are provided in Tables A4 through A25. These calculations demonstrate that some bridge decks may be under-designed based on moment calculations, cracking control calculations, or both. However, as observed previously, diaphragms are consistently located below the concrete deck dam repairs, providing substantial support at that location. This causes the assumption of the analysis to determine positive and negative bridge deck moments to be extremely conservative. It is therefore concluded that the cracking is not the result of inadequate structural steel reinforcing bars.

2.2.2 Construction Specifications

As noted previously, the most significant change to PennDOT and District 3-0 construction specifications was the addition of AAA-P and HPC concrete mixtures. After the present study began, PennDOT specified a revised concrete mixture for use exclusively on bridge decks designated as the AAA-P mixture. These mixtures are not considered in detail during this chapter because they were not in existence during the bridge deck dam rehabilitations reviewed in this study. In the following section, a review of relevant (to bridge deck cracking) sections of *PennDOT Specifications Publication 408* is provided.

2.2.2.1 Section 703: Aggregate

PennDOT Specifications Publication 408, Section 703, states that the fine aggregate should be Type A natural sand and should be either crushed or glacial while its fineness modulus (FM) shall not vary by more than ±0.20. These requirements are similar to AASHTO, as well as ASTM specifications (shown in Table 2-3). The FM is defined (ASTM C 125, Mindess et al. 2003) as the sum of the cumulative percentages retained on the standard sieves between the numbers four (#4) and one-hundred (#100) divided by 100. The fine aggregate gradation is known to have a large impact on the workability of concrete by affecting the water demand as well as the particle packing of aggregates, which determines the required paste content (Neville 1995, Mindess et al. 2003). Although the FM is a rough estimate of the consistency across mixtures, its simplicity of evaluation provides to concrete suppliers a basis for quality control of concrete and its workability (Mindess et al. 2003). It is recommended that this practice be continued.

Coarse aggregates are stated to be durable crushed stone or gravel and adhere to requirements listed in Table 2-4. Blast furnace slag, steel slag, or granulated slag aggregates may be permitted as well. According to Section 704 of the specifications, only the following coarse aggregate gradations are allowed for cement concrete: AASHTO #57, #67, or #8.

Table 2-3: Requirements for fine aggregate gradations based on PennDOT and ASTM C 33-11 specifications

	Percent passing each sieve	
Sieve #	PennDOT	ASTM C 33-11
3/8-in	100	100
No. 4	95-100	95-100
No. 8	70-100	80-100
No.16	45-85	50-85
No. 30	25-65	25-60
No. 50	10-30	5-30
No. 100	0-10	0-10
No. 200	0-3	0-3
Fineness Modulus	2.30-3.15	2.30-3.10

Table 2-4: Quality requirements for coarse aggregates used in concrete bridge decks in Pennsylvania

Criteria	PennDOT	ASTM C33-11
$MgSO_4$ Soundness, Max%	10	18
LA Abrasion Loss, Max %	45	50
Thin and elongated particles, Max %	15	---
Crushed fragments, Min %	55	---
Compact density (lb/ft^3)	70	---
Deleterious shale, Max %	2	5.0
Clay lumps, Max %	0.25	---
Friable particles excluding shale, Max%	1.0	3.0 (includes clay lumps)
Coal or coke, Max %	1	0.5
Glassy particles, Max %	4	---
Iron, Max %	3	---
Total of deleterious shale, clay lumps, friable particles, coal or coke allowed, Max. %	2	5.0

The most commonly used coarse aggregates in PennDOT bridge projects are the AASHTO #57 and #8. Larger MSAs (maximum size of aggregate) need less mixing water (or paste content) to achieve a satisfactory workability (Walker and Bloem 1960). On the other hand, a smaller MSA may results in a greater compressive strength (Walker and Bloem 1960). With respect to this research, #8 limestone aggregates may be better suited as their smaller size allows for proper concrete flow around closely placed reinforcing bars. This would allow the concrete to adhere to ACI guidelines with respect to concrete cover thickness and clear spacing between reinforcing bars. Blending of #8 coarse aggregates with aggregates ASTM #89 and/or #9 may provide a better aggregate packing and reduce the paste content, according to the Shiltstone method (Shiltstone 1990). This will be further discussed below. As mentioned in Task 1, paste content has a significant impact on the cracking tendency of concrete. It is not recommended to use AASHTO #10 aggregates due to the introduction of too many fine particles less that #100 sieve. Table 2-5 presents aggregate gradations according to AASHTO (same as PennDOT) and ASTM.

Table 2-5: Aggregate gradations for AASHTO (A) and ASTM C 33-11 (C) coarse aggregate.

	Percent passing each sieve					
Sieve #	**A(#57)**	**A(#67)**	**A(#8)**	**C(#89)**	**C(#9)**	**A(#10)**
1.5 in	100	-	-	-	-	-
1 in	90-100	100	-	-	-	-
¾ in	-	90-100	-	-	-	-
½ in	25-60	-	100	100	-	-
3/8 in	-	20-55	85-100	90-100	100	100
No.4	0-10	0-10	10-30	20-55	85-100	85-100
No.8	0-5	0-5	0-10	5-30	10-40	-
No. 16	-	-	0-5	0-10	0-10	-
No. 50	-	-	-	0-5	0-5	-
No. 100	-	-	-	-	-	10-30

<u>2.2.2.2 Section 704: Cement Concrete</u>

PennDOT Specifications Publication 408, Section 704, specifies the mixture requirements for the AAA concrete mixture, as well as the AA, A, C, and HES concrete mixtures. Although AAA#57 concrete (i.e., a AAA concrete mixture using #57 coarse aggregates) was implemented exclusively on the three past projects reviewed during this study, AAA#8, AA, and HPC#57 concretes have also been used during construction of some rehabilitation projects outside those reviewed here. Table 2-6 provides concrete performance criteria for use in bridge decks and other structures (note that the HPC mixture is not directly specified in Publication 408).

Table 2-6: PennDOT's cement concrete criteria for bridge decks and other structures

Class of Concrete	Use	Cement Factor (lbs./yd^3)		Max allowable w/cm	Min Design Compressive Strength (psi)			Proportions Coarse Aggregate Solid Volume (ft^3/yd^3)	28-Day Structural Design Comp. Strength (psi)
					Days				
		Min	Max		3	7	28		
AAA	Bridge decks	634.5	752	0.43	-	3600	4500	-	4000
AAA-P	Bridge Decks	560	752	0.45	-	3000	4000	-	4000
AA	Structures/Misc.	587.5	752	0.47	-	3000	3750	9.93-13.10	3500
A	Structures/Misc.	564	752	0.50	-	2750	3300	10.18-13.43	3000
HES	Structures/Misc.	752	846	0.4	3000	-	3750	9.10-12.00	3500

The specification requirements for AAA concrete show the minimum and maximum cement factors of 634.5 lbs./yd^3 (6¾ sacks) and 752 lbs./yd^3 (8 sacks), respectively. The allowable cement factors can be as high as 800 lbs./yd^3 (8½ sacks), 846 lbs./yd^3 (9 sacks), or 752 lbs./yd^3 (8 sacks) for AA, HES, and AAA-P concretes, respectively. The maximum allowable cement factor limit can be waived if pozzolan is added to the mix, provided the Portland cement portion does not exceed the specified cement factor. In such cases, the cement paste content of concrete may be even higher than those values specified in Table 2-6. Section 1001 specifies that the cement factor could be increased even further, with written consent of the PennDOT representative, to obtain high early-strength concrete. The specified cementitious materials contents are excessive when compared with literature recommendations and can significantly increase the risk of early-age cracking in restrained concrete elements (e.g., bridge decks). The maximum recommended cementitious materials content to prevent cracking has been reported in literature as 611 to 725 lbs./yd^3 (6½ to 7¾ sacks). More recently, however, McLeod et al. (2009) found these recommendations to be too high and suggested limiting the cement factor to between 500 and 540 lbs./yd^3 (<5¾ sacks). There is a strong positive relationship between concrete cracking and increased cement content (Krauss and Rogalla 1996, Schmitt and Darwin 1999, Saadeghvaziri and Hadidi 2005) as discussed in detail in chapter 1, since it is the cement paste phase in concrete that shrinks and also causes thermal contraction as the heat generated during cement hydration dissipates to the ambient.

2.2.2.2.1 Suggested Modifications to PennDOT Specifications

It is recommended that PennDOT reduce the minimum and maximum allowable cement factor for these aforementioned concretes. This requires designing more efficient aggregate blends to be able to increase the overall aggregate content and reduce the cement paste content of concrete. A method for improving the aggregate gradation in concrete has been suggested by Shiltstone (1990), and has been shown to considerably reduce the cement content of concrete. PennDOT may consider implementation of the Shiltstone method to improve its concrete mixtures. In such case, blending of coarse aggregate gradations other than AASHTO #57, #67, or #8 should be permitted.

The minimum design compressive strength of the concrete mixtures compare well with the literature provided in Task 1 for bridge deck design. However, literature (e.g., Frosch et al. 2003) also recommends enforcing a maximum compressive strength to prevent the use of excessively strong and stiff concretes that lead to very large shrinkage and thermal stresses (the causes are discussed in chapter 1 of this report). PennDOT may consider recommending or enforcing a maximum allowable compressive strength at 1000 psi above the concrete 28-day structural design compressive strength.

The maximum allowable slump of concrete, according to Section 704 of the PennDOT specifications is 8 inches when high range water-reducing admixtures have been used. In contrast with literature recommendations of 2 to 4 inches, the PennDOT slump allowance is excessive and can contribute to settlement cracking as described in chapter 1.

<u>2.2.2.3 Section 711: Concrete Curing Material and Admixtures</u>

PennDOT Specifications Publication 408, Section 711, discusses the criteria for acceptable polyethylene sheeting, burlap- or fiber-backed sheeting, burlaps, insulating mats, and foam insulation that can be used for proper curing of concrete. For bridge decks, only a double thickness of burlap should be used as a cover material. Criteria for curing compounds are also included. The section also includes specifications for chemical admixtures to be used for concrete. The required method and duration of moist curing is not covered here, but in Section 1001.

2.2.2.4 Section 724: Pozzolans

PennDOT Specifications Publication 408, Section 704, permits the use of pozzolans in addition to or as a replacement for Portland cement. It does not, however, allow the use of ternary concrete mixtures incorporating both slag and fly ash. If fly ash is used (both class C and F are permitted), the Portland cement portion may be reduced by a maximum of 15% (note that this may not be sufficient to mitigate alkali silica reaction if aggregates are potentially reactive (Thomas 2011)). If ground granulated blast furnace slag (GGBFS) is used, the Portland cement portion may be reduced by a minimum of 25% to a maximum of 50%. The slag must be grade 100 or higher. If silica fume is used, the allowable cement replacement level is 5% to 10%. When aggregates are considered potentially reactive (i.e., prone to alkali-silica reaction), indicated by an expansion higher than 0.10% at 14 days per AASHTO T303 test, a low alkali cement (equivalent alkali content < 0.60%), or a combination of cement and pozzolan must be used.

Section 724 of *Publication 408* provides requirements for GGBFS, fly ash, or silica fume for use in concrete. Only GGBFS grades 100 or 120 should be used. GGBFS should adhere to AASHTO M 302. Fly ash should adhere to AASHTO M 295 for classes C, F, and N with a limit on the loss of ignition (LOI) of 6.0%. Silica fume should adhere to AASHTO M 307.

2.2.2.5 Section 1001: Cement Concrete Structures

PennDOT Specifications Publication 408, Section 1001, provides requirements for construction of reinforced concrete structures, including proper formwork (both wooden and metal pans), rebar installation, concrete placement (including pumping), consolidation and finishing, and concrete curing. In the following, a summary of specifications that may impact the cracking risk of concrete is provided. Specific provisions applicable to bridge deck concrete are emphasized.

GENERAL PROVISIONS: The specifications require the contractor to submit, for review and acceptance, a QC Plan showing the methods, sequence, and schedule for placing concrete at least 15 days before the start of construction of the element of work. During concrete placement, segregation must be prevented; for example, concrete must not be dropped from a distance greater than 4 ft. Concrete must not be placed upon frozen foundation material, in forms containing frost, around frosted reinforcement, or in pile shells surrounded by ice or frozen earth. Adjacent batches of concrete shall not be placed if their temperature differs by more than 20°F. For succeeding batches, concrete must be placed in the forms within 30 minutes. If the construction plane is horizontal and concrete placement is stopped for more than 30 minutes, acceptable keyways and sufficient dowel bars must be provided. In areas where reinforcement extends through a construction joint, concrete adjacent to previously placed concrete shall not be placed until at least 24 hours has elapsed. Concrete must be placed in horizontal layers no more than 15 inches in depth. Each part of the form must be filled by depositing the concrete as close to its final position as possible. Concrete should not be worked along the forms from the point of deposit. Concrete placement should not displace the reinforcement.

Mechanical vibrators should be used to properly consolidate the concrete. The vibrator should not be attached to the forms or reinforcement and should be applied to the concrete, at intervals not exceeding 3 ft, immediately after the concrete has been deposited. The vibrator should be properly moved throughout the mass, completely working the concrete around the reinforcement and other embedded fixtures, and into the corners and angles of the forms. Any reinforcement displacement caused by the vibrator must be corrected before continuing vibration. The vibrator should be moved slowly to prevent segregation and must not be used to spread the concrete. Concrete must be properly finished using approved methods and equipment. At the time of finishing, water or a curing agent cannot be added to the concrete surface to assist in finishing. **Curing must begin as soon as the concrete has been placed and is sufficiently hardened**. If the curing temperature drops below 50°F at any time during a day, that day must not be counted as a curing day. This is applicable to bridge decks for curing during days 1-7. During days 8-14, a day during which the curing temperature drops below 40°F at any time during the day, should not be counted as a curing day. If at any time during the curing period, the curing temperature falls below 35°F, the Department will consider the work unsatisfactory and will reject it. Curing temperature is the temperature of the air immediately adjacent to concrete. Where concrete is not covered by forms or other protective coverings, or where protective coverings are considered inadequate, the curing temperature will be the air temperature. High-low thermometers must be used to maintain an accurate daily record of air and curing temperatures during cool and cold weather. These temperature records are submitted daily to the Inspector-in-Charge.

PROVISIONS SPECIFIC TO BRIDGE DECK CONSTRUCTION: Specifically for bridge decks, a deck pre-placement meeting must be scheduled at least 2 weeks before concrete deck placement, to review the specification, method and sequence of placing deck concrete, quality control testing, and method of protective measures, to **control the concrete evaporation rate**. Bridge deck concrete must only be placed if the temperature of concrete is within 50 to 80^{o}F (up to 90°F concrete can be used for approach slabs). **The water evaporation rate from the surface of concrete must be determined before starting the deck placement and every hour during the placement. An evaporation rate of 0.15 lb/ft^{2}hr must not be exceeded.** The allowable evaporation rate for exposed finished concrete is determined by ACI 305R-91, Figure 2.1.5. All remediation equipment and procedures [to reduce water evaporation rate] must be readily available at the bridge deck placement site, as submitted and approved at the deck pre-placement meeting before starting the placement. **If the value is exceeded, concrete placement must be stopped until protective measures are taken to reduce the values to an acceptable level**. Fog cure misting is an acceptable method to mitigate an excessive evaporation rate (the specifications provide a description of acceptable equipment to be used for fog curing). The fog must be applied over the entire placement that is not covered by wet burlap. **Concrete must not be left exposed for extended duration** (this is an ambiguous statement as it is unclear how long should be considered "extended duration"). Concrete must be placed 5 to 8 feet ahead of finishing machine to prevent any premature concrete drying.

Concrete must be placed from the center of the span (i.e., positive moment areas) toward each leg or abutment simultaneously. Unless allowed in writing by the District Executive, truck mixers, truck agitators, or other heavy motorized equipment must not be allowed on the deck spans in which concrete is being placed. Concrete must be placed at a minimum rate of 20 linear feet of deck per hour, in a longitudinal direction, except for reinforced concrete slabs and rigid frames. Concrete must be vibrated to prevent honeycombing and voids, especially at construction joints, expansion joints, valleys, and ends of form sheets. The specifications include language for allowable vibrating screeds, strike-off finishing machines and methods, as well as manual final finishing, straight-edge testing, and texturing operations.

CURING: **Immediately after texturing operations are completed, intermediate curing must be performed by applying a monomolecular film curing agent** (e.g., Confilm) **to prevent surface drying before placement of curing covers**. This has to be performed immediately after the final finishing operation is completed on any area (surface should not be further disturbed/finished after application of the monomolecular film). **Water curing has to be performed** using a fog-spray, perforated pipe or hose watering system **to keep forms and curing covers saturated during the curing period (14 days minimum)**. For bridge decks, use only a double thickness of burlap as a cover material. **Deck must be water cured (as opposed to use of a curing compound) for a minimum of 14 days** by maintaining wet burlap application within 10 ft to 18 ft behind the finishing equipment at all times. Curing must continue until minimum compressive strength of concrete is attained (using molded cylinder specimens), but not less than 14 days. The minimum compressive strength is 3000 psi for AAA-P concrete and 3300 psi for AAA concrete. For bridge decks placed between September 1 and March 1, a penetrating sealer as specified in Section 1019.3(c) 2 must be applied. In cold temperatures, newly placed concrete must be insulated and heated if necessary to maintain its temperature above 50°F (during the first 7 days) or above 40°F (during days 8-14) but not more than 80^{o}F. Live construction loads should not be allowed on the deck before 7 days and a minimum compressive strength of 3250 psi. Bridge deck may be opened to traffic after a period of 14 days after placing the last deck concrete and the deck concrete has attained a minimum compressive strength of 4000 psi.

The bridge approach slab can be cured using white membrane forming curing compound if the air temperature is above 40°F. In this case, the curing compound must be applied immediately after completion of the finishing and after the surface film of water has disappeared, while the surface is still damp. If normal curing is delayed, an intermediate monomolecular film curing agent must be applied. Curing compound must be sprayed with a minimum coverage of 1 gal per 150 sq ft of concrete. Curing materials must be maintained for at least 96 hours.

2.2.2.5.1 Suggested Modifications to PennDOT Specifications

Many of the existing provisions discussed above properly state practices for mitigation of early-age cracking of concrete. It is very important to ensure these practices are enforced during construction operations. In addition, there are certain provisions that could be modified or better clarified to guide contractors in construction of high quality concrete bridge decks. These are listed below.

- Plastic shrinkage cracking is known to be directly related to the evaporation rate of bleed water from the surface of fresh concrete (Wittman 1976, Cohen et al. 1990, Radocea 1994). Therefore, it is important to ensure that rate of evaporation remains less than rate of bleeding so the surface of concrete never dries (ACI 308R 2001). The typical water bleeding rate of concrete is between 0.1 to 0.2 lbs./ft^2/hr. Therefore, it may be justified for PennDOT to reduce the allowable evaporation rate from the existing limit of 0.15 lbs./ft^2/hr. to 0.1 lbs./ft^2/hr. It is also very important to make sure that the concrete temperature, air temperature, humidity, and wind speed are monitored throughout the placement of concrete, the estimated evaporation rate is calculated, and proper remediation techniques are available for immediate application if the evaporation rate exceeds 0.1 lbs./ft^2/hr.

- Concrete surface must not be left exposed for extended duration. The specifications must clearly mention that concrete must be protected (e.g., using fog spray) if the placement and finishing operations have to be temporarily stopped before the final finishing. Given that the current specifications prohibit the use of intermediate curing agent before final finishing, in such cases, the surface could be totally exposed without any protection against moisture evaporation. It can be recommended to apply the first layer of pre-soaked burlap 10 minutes after strike-off and apply a second layer within 5 minutes (McLeod et al. 2009). Continuation of moist curing for 14 days is well recommended.

- Approach slabs are reinforced and are prone to a similar level of cracking risk as the bridge deck. They are however, subject to a more relaxed standard of curing according to the current specifications. It is recommended that PennDOT adopts similar moist curing requirements for bridge approach slabs.

2.2.2.6 Sections 709 and 1002: Reinforcement Bars

PennDOT Specifications Publication 408, Section 709, specifies the requirements on material properties of different types of reinforcing steel for use in concrete. Specifications for epoxy coating are also included. Section 1002 includes requirements for bar splicing and lapping, methods for fabrication and installation of rebar cages, proper rebar storage, bending and straightening requirements, placing and fastening, and rebar support (chair) system. The specification states the need to tie all bars at intersections and not weld them. There are no requirements on the maximum rebar size or spacing for concrete bridge decks. Literature presented in chapter 1 states that reinforcing bars shall be no greater than #5 and clear bar spacing should be limited to 6 inches. However, AASHTO LRFD Design Specifications do not place a limit on the spacing of the reinforcing bars. Also, AASHTO does not place a limit on the size of the reinforcing bars. However, for all practical applications, the #6 size reinforcing bar shall be the minimum reinforcing bar size.

2.2.2.7 Section 1040: Concrete Bridge Deck Repair

PennDOT Specifications Publication 408, Section 1040, states that basic patching should be done in accordance with Section 704 and should use concrete mixture AAA#8. Also, all pre-existing reinforcing bars should be sandblasted, epoxy coated, and readied for reuse. In practice, if a current bar is damaged during demolition, it is removed and a new reinforcing bar is added in its place and tied at all reinforcing bar intersections.

2.3.0 REVIEW OF PAST PROJECTS

This section of this report reviews PennDOT District 3-0 projects 15-7PP, 180-044, and 180-058. These projects were completed before 2010. For these three projects, early-age cracking on concrete adjacent to the bridge deck dam rehabilitations was observed within a few weeks of construction. PennDOT subsequently placed an epoxy-coated sealant in order to prevent moisture and deicing salt ingress through the cracks. This section reviews the material properties of the concrete placed on these jobs and compares it to the literature recommendations and existing PennDOT specifications. No data was recorded or was available to the PSU team for modulus of elasticity, Poisson's ratio, heat of hydration, coefficient of thermal expansion, thermal diffusivity, and rapid chloride penetration of concrete as well as the relative humidity, average wind speed on site during concrete placement and finishing operations. Concrete cracking occurred inconsistently across the bridges reviewed in this study. On a single bridge deck, cracking may have occurred on concrete placed during phase A or phase B, or both. Also, some decks did not exhibit any cracking. Due to the application of modified concrete overlays, it was not possible to investigate all the cracking that occurred on the bridge decks.

2.3.1 Project 15-7PP

PennDOT District 3-0 bridge deck dam rehabilitation project 15-7PP provides rehabilitation for bridges in Tioga and Lycoming Counties. Tioga County bridges are along US Route 15 at Section 7PR over PA State Route 414 as well as PA State Route 287 at Section 7PM over Wilson Creek. Lycoming County bridges are along US Route 15 at Section 7PQ over T-811 and Beck Run as well as US Route 15 at Section 7PP over PA State Route 284. For this project, AAA#57 concrete was used. Table 2-7 presents detailed information gathered from the project and compares it with information gathered from PennDOT specifications and published literature. As discussed before, PennDOT specifications' allowance on cement content and slump are too high in comparison with literature recommendations. The absence of specifying a maximum allowable compressive strength has resulted in the use of concrete with 23% higher strength than required (5530 psi as opposed to 4500 psi). Information about concrete temperature, ambient humidity and wind speed, and estimated evaporation rate of water from the surface of fresh concrete were not available in concrete construction documentations provided to the PSU team. According to PennDOT specifications, these parameters must be monitored and proper remediation techniques must be employed to prohibit plastic shrinkage cracking. The duration of water curing is unknown, but should be at least 14 days according to PennDOT specifications.

Table 2-7: Data for Project 15-7PP

	15-7PP Data	PennDOT Specifications	Literature Recommendations	Comments
w/cm Ratio	0.42	<0.43	0.42 to 0.45	OK
Cementitious Materials Content (lbs/yd^3)	658	634.5 to 752	<540	High
Cement Paste Fraction	0.35	0.31 to 0.43	<0.35	Borderline
Design Air Content (%)	6.0	6.0	--	--
Measured Air Content (%)	4.9 - 7.0	6.0±1.5	--	OK
Mineral Admixtures	35% Slag as replacement of Portland cement	Slag or Fly ash	Slag is a suitable admixture	OK
Chemical Admixtures	Water reducer, set retarder, air entrainer	--	--	--
Cement Type	Type I	Type I	Type I or II	OK
Coarse Aggregate Type	AASHTO #57	AASHTO #57, #67, or #8	Proper aggregate blending to improve packing and minimize paste content	OK
Fine Aggregate Type	PennDOT Type A	PennDOT Type A	ASTM C 33	OK
Design Slump (in)	4.0	8 Max	2.0 to 4.0	OK
Measured Slump (in)	2.5 - 5.5	±1.5 in of design slump	--	OK
28-day Structural Compressive Strength (psi)	4000	4000	4000	OK
28-day Minimum Design Compressive Strength (psi)	4500	4500	--	--
Lab Tested 7-day Compressive Strength (psi)	3710	--	--	--
Lab Tested 28-day Compressive Strength (psi)	5530	4500	--	High
Ambient Air Temperature	Low of 65°F and high of 84°F	--	45°F to 90°F	OK
Concrete Temperature at Time of Placement (°F)	Not available	50°F to 80°F Must be measured on first 3 consecutive trucks	<80°F	Potential violation of specs
Curing	Wet burlap	Wet burlap	Fog/burlap	OK
Duration of Moist Curing (days)	Not available	14 days	Min 7 days	??

Ambient Humidity, Wind Speed	Not monitored or not available	Must be regularly monitored on site to ensure water evaporation rate is acceptable	Must be regularly monitored on site to ensure water evaporation rate is acceptable	Potential violation of specs
Evaporation of Bleed Water (lbs/ft^{2}hr)	Not monitored or not available	<0.15	0.1 to 0.2	Potential violation of specs
Reinforcing Bar Size	#5 and #6	#5 and #6	#5 max	#6 may be too large
Reinforcing Bar Spacing (in)	Sometimes more than 6in	3ft Max	6	Spacing may be too far
Cover Thickness (in)	2 to 3	--	2 to 3	OK

2.3.2 Project 180-044

Project 180-044 provides rehabilitation for bridges in Lycoming County along PA State Route 180 at Section 044 over T-852, over PA State Route 2045, and over PA State Route 2075 and PA State Route 87. For this project, AAA#57 concrete was used. Table 2-8 presents detailed information gathered from the project and compares it with information gathered from PennDOT specifications and published literature. Similar to the previous project, the absence of specifying a maximum allowable compressive strength has resulted in the use of concrete with 22% higher strength than required (5510 psi as opposed to 4500 psi). Information about concrete temperature, ambient humidity and wind speed, and estimated evaporation rate of water from the surface of fresh concrete were not available in concrete construction documentations, which could be a potential violation of PennDOT specifications. The duration of water curing is unknown; but should be at least 14 days according to PennDOT specifications. In addition, the measured air content of concrete was occasionally outside the limits allowed by PennDOT.

Table 2-8: Date for Project 180-044

	180-044 Data	PennDOT Specifications	Literature Recommendations	Comments
w/cm Ratio	0.41	<0.43	0.42 to 0.45	OK
Cementitious Materials Content (lbs/yd^3)	658	634.5 to 752	<540	High
Cement Paste Fraction	0.35	0.31 to 0.43	<0.35	Borderline
Design Air Content (%)	6.0	6.0	--	--
Measured Air Content (%)	4.8 – 8.5	6.0±1.5	--	High
Mineral Admixtures	35% Slag as replacement of Portland cement	Slag or Fly ash	Slag is a suitable admixture	OK
Chemical Admixtures	Water reducer, set retarder, air entrainer	--	--	--
Cement Type	Type I	Type I	Type I or II	OK
Coarse Aggregate Type	AASHTO #57	AASHTO #57, #67 or #8	Proper aggregate blending to improve packing and minimize paste content	OK
Fine Aggregate Type	PennDOT Type A	PennDOT Type A	ASTM C 33	OK
Design Slump (in)	4.0	8 Max	2.0 to 4.0	OK
Measured Slump (in)	3.5 - 5.5	±1.5 in of design slump	--	OK
28-day Structural Compressive Strength (psi)	4000	4000	4000	OK
28-day Minimum Design Compressive Strength (psi)	4500	4500	--	--
Lab Tested 7-day Compressive Strength (psi)	3800	--	--	--
Lab Tested 28-day Compressive Strength (psi)	5510	4500	--	High
Ambient Air Temperature	Low of 45°F and high of 67°F	--	45°F to 90°F	OK
Concrete Temperature at Time of Placement (°F)	Not available	50°F to 80°F Must be measured on first 3 consecutive trucks	<80°F	Potential violation of specs
Curing	Wet burlap and plastic	Wet burlap	Fog/burlap	OK
Duration of Moist Curing (days)	Not available	14 days	Min 7 days	??

Ambient Humidity, Wind Speed	Not monitored or not available	Must be regularly monitored on site to ensure water evaporation rate is acceptable	Must be regularly monitored on site to ensure water evaporation rate is acceptable	Potential violation of specs
Evaporation of Bleed Water (lbs/ft^2hr)	Not monitored or not available	<0.15	0.1 to 0.2	Potential violation of specs
Reinforcing Bar Size	#5 and #6	#5 and #6	#5 max	#6 may be too large
Reinforcing Bar Spacing (in)	Sometimes more than 6in	3ft Max	6	Spacing may be too far
Cover Thickness (in)	2 to 3	--	2 to 3	OK

2.3.3 Project 180-058

Project 180-058 provides rehabilitation for bridges in Lycoming County along PA State Route 180 at Section 58M over PA State Route 2029, over PA State Route 2014, over Miller's Run and T-480. For this project, AAA#57 concrete was used. Table 2-9 presents detailed information gathered from the project and compares it with information gathered from PennDOT specifications and published literature. Similar to the previous projects, the absence of specifying a maximum allowable compressive strength has resulted in the use of concrete with 27% higher strength than required (5720 psi as opposed to 4500 psi). Information about concrete temperature, ambient humidity and wind speed, and estimated evaporation rate of water from the surface of fresh concrete were not available in concrete construction documentations, which could be a potential violation of PennDOT specifications. The duration of water curing is unknown, but should be at least 14 days according to PennDOT specifications. In one occasion, the measured air content of concrete was outside the limits allowed by PennDOT.

Table 2-9: Data for Project 180-058

	180-058 Data	PennDOT Specifications	Literature Recommendations	Comments
w/cm Ratio	0.42	<0.43	0.42 to 0.45	OK
Cementitious Materials Content (lbs/yd^3)	658	634.5 to 752	<540	High
Cement Paste Fraction	0.35	0.31 to 0.43	<0.35	Borderline
Design Air Content (%)	6.0	6.0	--	--
Measured Air Content (%)	5.5 - 7.0 (one at 9.0)	6.0±1.5	--	OK
Mineral Admixtures	35% Slag as replacement of Portland cement	Slag or Fly ash	Slag is a suitable admixture	OK
Chemical Admixtures	Water reducer, set retarder, air entrainer	--	--	--
Cement Type	Type I	Type I	Type I or II	OK
Coarse Aggregate Type	AASHTO #57	AASHTO #57, #67, or #8	Proper aggregate blending to improve packing and minimize paste content	OK
Fine Aggregate Type	PennDOT Type A	PennDOT Type A	ASTM C 33	OK
Design Slump (in)	4.0	8 Max	2.0 to 4.0	May be high
Measured Slump (in)	3.5 - 5.0	±1.5 in of design slump	--	OK
28-day Structural Compressive Strength (psi)	4000	4000	4000	--
28-day Minimum Design Compressive Strength (psi)	4500	4500	--	--
Lab Tested 7-day Compressive Strength (psi)	4170	--	--	--
Lab Tested 28-day Compressive Strength (psi)	5720	4500	--	High
Ambient Air Temperature	Low of 51°F and high of 78°F	--	45°F to 90°F	OK
Concrete Temperature at Time of Placement (°F)	Not available	50°F to 80°F Must be measured on first 3 consecutive trucks	<80°F	Potential violation of specs
Curing	Wet burlap	Wet burlap	Fog/burlap	OK
Duration of Moist Curing (days)	Not available	14 days	Min 7 days	??

Ambient Humidity, Wind Speed	Not available or not monitored	Must be regularly monitored on site to ensure water evaporation rate is acceptable	Must be regularly monitored on site to ensure water evaporation rate is acceptable	Potential violation of specs
Evaporation of Bleed Water (lb/ft^{2}hr)	Not available or not monitored	<0.15	0.1 to 0.2	Potential violation of specs
Reinforcing Bar Size	#5 and #6	#5 and #6	#5 max	#6 may be too large
Reinforcing Bar Spacing (in)	Sometimes more than 6in	3ft Max	6	Spacing may be too far
Cover Thickness (in)	2 to 3	--	2 to 3	OK

2.4.0 REVIEW OF ACTIVE PROJECTS

A PennDOT District 2-0 bridge rehabilitation project was visited three times in May 2012 to observe construction activities on bridge deck dam replacements. The project was located on state route 322 (SR 0322) at section Z08 in Derry and Brown Townships in Mifflin County (ECMS# 4722). Another bridge dam rehabilitation was observed in District 3-0 in July 2012. This project was located in Sullivan County along state route 87 at Section 65M over Little Loyalstock Creek (ECMS# 82195). This section provides the observations from the three site visits in District 2-0 and the one site visit in District 3-0.

2.4.1 District 2-0: First Site Visit (05/17/2012)

PSU and QES representatives met with Mr. Nicholas Minarchick, PennDOT's project manager, at the project office on 05/17/2012 and discussed the site visit plan. The team visited a bridge deck dam that had been recently replaced with concrete being placed the previous day (Figure 2-1). The approach slab (right of the joint) was finished to grade while the deck (left of the dam) was finished roughly to enhance bonding to the final latex modified concrete overlay to be applied later. The deck concrete was also finished 1.5 inches below final grade. A latex-modified overlay will consist of the extra 1.5 inches needed to meet grade. It was observed that the approach slab was being cured using a curing compound and not using water curing with double wet burlap. This practice is allowable by PennDOT specifications, which treats the approach slab as a concrete pavement which is subject to less strict standards of curing. Given that the approach slab is reinforced, the research team recommends that PennDOT reconsider the specifications for curing of bridge approach slabs. The team also observed the concrete placement of the approach slab on the other end of the bridge (Figure 2-2). The concrete was placed, vibrated, finished, and then cured using a curing compound instead of wet burlap.

The deck concrete was being cured with burlap that was totally dry (note that the concrete was only 1 day old). This is in violation of PennDOT specifications, which require the use of fog-spray, perforated pipe, or hose watering system to keep curing covers (i.e., burlaps) saturated during the entire curing period of 14 days. To minimize the risk of early-age cracking of concrete, it is very important that PennDOT water curing specifications are strictly enforced.

Figure 2-1: A completed bridge deck dam replacement

The team then visited dam rehabilitation activities on a different structure within the same project. Demolition and removal of the concrete dams occurred prior to the visit; Figure 2-3 presents the final product after demolition. The existing reinforcing bars are water blasted to remove any residual concrete and epoxy coated manually.

Figure 2-2: Vibrating the approach slab on the opposite end of the deck

New dam hardware is shown in Figure 2-4, ready to be installed. The dam consists of longitudinal and transverse reinforcing bars as well as shear keys to allow for the proper transfer of load between the composite sections. It can be seen that the dam consisted of two sections to be welded together during installation. Both parts of the new dam were installed by lowering the hardware into place (Figure 2-5). The two sections were then welded together, with the welded area being sprayed with an epoxy bonding compound for protection (Figure 2-6). This was the last activity of the day for this dam. The contractor's crew was also working on preparing a second dam for replacement on this day. The second bridge consisted of five spans with discontinuous girders. This bridge's steel diaphragms were to be replaced with concrete diaphragms not to be placed concurrently with the deck's concrete.

Figure 2-3: Repair area of bridge deck with old dam and concrete removed

Figure 2-4: New dam hardware

Figure 2-5: New dam hardware after installation

Figure 2-6: Welded dam sections

2.4.2 District 2.0: Second Site Visit (05/18/2012)

The same project was visited by the team on 05/18/2012. The main activity was installation of new reinforcing bars in the block-outs. Originally, the concrete was to be poured on that day; however, it was postponed due to unforeseen challenges during the dam installations. Figure 2-7 shows one side of a dam with the bottom layer of reinforcing bars installed. Figure 2-7 also presents a single upright green reinforcing bar (epoxy coated) that is a replacement for a reinforcing bar that was damaged during demolition or experienced cross section loss from corrosion. Figure 2-8 shows one dam side with a newly installed dam outfitted with existing as well as new steel reinforcements. The concrete placement that was scheduled for this day was postponed until 05/22/2012.

2.4.3 District 2.0: Third Site Visit (05/22/2012)

A third site visit to the SR 322 project site occurred on 05/22/2012. All new reinforcing bars were placed, the joint openings for two dams were covered, and the block-outs were ready for concrete placement. Figure 2-9 shows both sides of a dam with new reinforcement installed. It was observed that not all reinforcing bars are orthogonal at intersections or parallel in the same plane. This may be due to the need to reuse existing steel. PennDOT concrete mixture AAA#8

(AAA concrete with AASHTO#8 coarse aggregate) was specified for this project. Mixture proportions for this concrete are provided in Table 2-10. Note that the cement paste content is larger than literature recommendations but does comply with PennDOT specifications. Plasticizing, air entraining, and set retarding admixtures were used in this mixture.

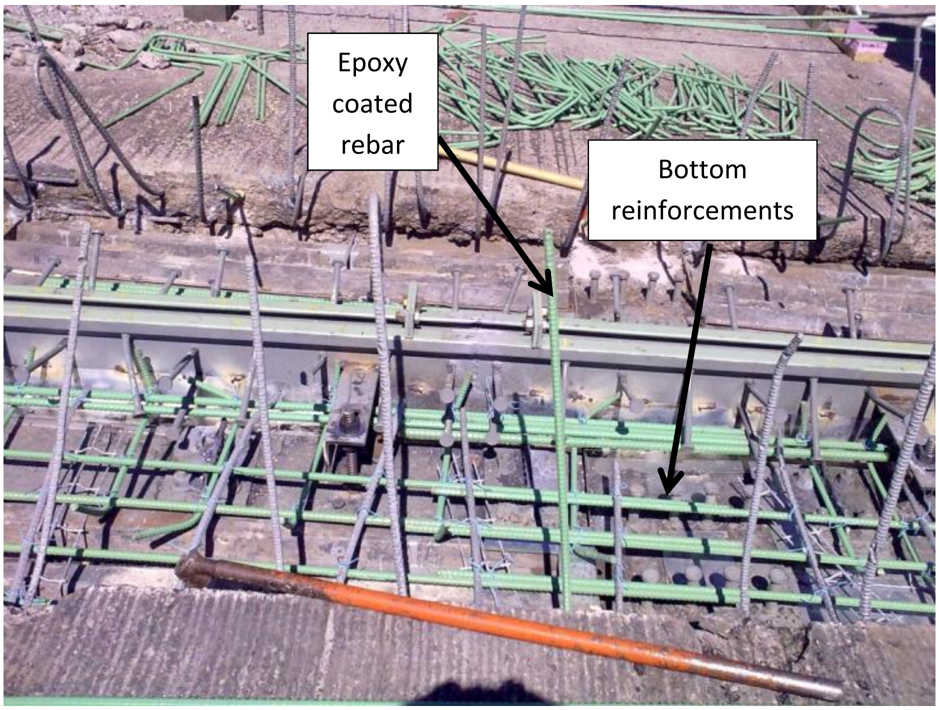

Figure 2-7: Bottom reinforcement bars installed on one side of the dam

Three concrete trucks were ordered by the contractor (Glenn O. Hawbaker). Juniata Concrete supplied each concrete batch. The first truck got stuck near the approach slab and was rejected due to too much time elapsing from the initial mixing time. The fresh properties of the concrete (slump and air content) were measured upon arrival of the second concrete truck. Figure 2-10 shows air content and slump tests being conducted on the samples and Table 2-11 shows the fresh concrete property test results for two trucks.

Figure 2-8: A dam with all new reinforcement installed

Figure 2-9: Both sides of a dam with new reinforcement installed

Figure 2-10: Fresh concrete properties tests. Left: air content test; Right: slump test

Table 2-10: Mixture proportions for AAA#8 used in PennDOT Project S.R. 322 (208) in Mifflin County (aggregate weights are based on SSD)

	AAA#8			
	Proportions by Volume (ft^3)	Proportions by Weight (lbs/yd^3)	PennDOT Specifications (lbs/yd^3)	Literature Recommendations (lbs/yd^3)
Cementitious Materials Content	3.70	706.0	634.5 to 752	<540
(Cement)	2.33	458.0	--	--
(GGBFS)	1.37	248.0	--	--
Water	4.66	291.0	--	--
w/cm	--	0.412	<0.43	0.42 to 0.45
Cement Paste Content (vol%)	0.37	---	0.31 to 0.43	<0.35
Coarse Aggregates AASHTO#8	9.79	1674.0	--	--
Fine Aggregates PennDOT Type A	7.23	1191.0	--	--
Design Air Content (%)	6.0	6.0	--	--
Design Slump (in)	4.0		8 Max	2.0 to 4.0
Total	27.0	3850.0	--	--

Table 2-11: Fresh concrete properties measured

	Second Truck	Third Truck	PennDOT Specs	Literature	Comments
Measured Air Content (%)	4.6	5.1	6.0±1.5	--	OK
Measured Slump (in)	2	4.25	4.0±1.5	--	Low for second truck
Concrete Temperature (°F)	78	78	50 to 80	< 80	OK but close to upper limit

The second truck's first air content test (4.2%) and slump test (2.25 in) failed *PennDOT Specifications Publication 408*. The mixer was allowed to rotate for approximately 50 more turns (no water addition was permitted by the inspector) after which an air content of 4.6% was achieved, qualifying the concrete to pass specification, while the slump remained below standard at 2.0 inches. Since the acceptance specification was based on air content while slump is accepted at the contractor's discretion, the PennDOT representative permitted the contractor to continue with the concrete placement. It was observed, however, that the fresh concrete was less workable than expected and the material did not finished as easily as the concrete from the third truck. The temperature of the concrete at placement was 78^{o}F, which is within the acceptable limits of PennDOT and literature. Care must be taken, however, to ensure concrete temperature does not exceed 80^{o}F in warmer months.

The ambient climatic conditions were recorded by the PSU team and are presented in Table 2-12. PSU members did not observe the contractor or PennDOT performing similar measurements, nor did they observe available remediation equipment and procedures at the site to make sure that the evaporation rate from the surface of concrete never exceeded 0.15 lb/ft^{2}hr. It is very important to enforce these regulations to eliminate the risk of plastic shrinkage cracking.

Figure 2-11 shows the placement and finishing of concrete from the second delivery truck. As the picture shows, the concrete was placed using a truck chute and then vibrated and finished. Vibration was noted to occur in one spot for a relatively long time. Figure 2-12 shows the rough finish of the first dam used for better adherence to the latex overlay. This rough finish and the subsequent 1.5-inch latex overlay reduced the concrete's top cover to only 1-inch at this phase of installation. For high slump concrete, this low value of rebar cover can result in settlement cracking over the rebar (Issa 1999).

Table 2-12: Ambient climatic conditions

Ambient Conditions	Project Value	PennDOT Specs	Literature
Minimum Temperature	78.6 °F	--	45 °F
Maximum Temperature	82.5 °F	--	90 °F
Minimum Relative Humidity	55.9%	--	--
Maximum Relative Humidity	68.0%, 10.0% greater at concrete surface	--	--
Maximum Wind Speed	4.0 mph	--	--
Estimated Evaporation Rate (lbs/ft^2hr)	0.05	<0.15	0.1 to 0.2

Figure 2-11: First dam’s pouring, vibration, and finishing

Figure 2-12: The rough finish of the first dam

The second concrete truck filled the first dam and a quarter of the second dam block-out area. The concrete delivered by the third truck passed the acceptance tests (4.25-in slump and 5.1% AC) and was used to complete the second expansion dam placement. This batch of concrete was observed to be more workable than the second batch. Figure 2-13 shows pouring and vibrating of the concrete delivered by the third truck. Figure 2-14 shows the rough finish of the second dam. The final top surface would become flush with the top of the dam hardware when a latex overlay of 1.5-inch thickness is placed. The finished surfaces were covered by wet burlap and white plastic sheeting. Figure 2-15 shows spreading of the burlap and Figure 2-16 shows the final cover. Final white cover was placed within 1.5 hours of initial concrete placement. The wet burlap was placed on top of the concrete approximately 30 to 40 minutes after final finish. This is considered late according to the literature, which suggests placing the first layer of wet burlap within 10 minutes and a second layer with 5 minutes afterward (McLeod et al. 2009). Also, the PSU team did not observe the use of an intermediate curing agent (monomolecular film) to make sure the surface of newly placed concrete is not exposed to evaporation. Application of such agent immediately after finishing is required by PennDOT specifications.

Water curing using the wet burlap would continue for 7 days. This is in violation of PennDOT specifications, which require 14 days of water curing. It is unclear how the contractor was to ensure that the burlaps would remain wet during the entire water curing period.

Figure 2-13: Placing and vibrating concrete from the third truck

Figure 2-14: The rough finish of the second dam

Figure 2-15: Spreading the wet burlap on the finished surface

Figure 2-16: Final white plastic sheeting cover

2.4.4 District 3-0: Site Visit (07/16/2012)

A PSU representative met with Mr. Paul King, PennDOT's Structural Control Engineer for District 3-0, at the district office on 07/16/2012 and discussed the site visit plan. The team then traveled to the site and met PennDOT quality control engineer, Mr. Chris Neyhart, as well as the contracting team of Glenn O. Hawbaker. The concrete was supplied by Centre Concrete and was a PennDOT AAA#8 concrete mixture (Table 2-13). Concrete was delivered using two truck mixers. The fresh properties of the two concrete trucks are provided in Table 2-14. These fresh properties passed the QC/QA tests; the data shows that all properties adhered to PennDOT specifications. GGBFS was used to replace 35% of the cement by weight. The bridge dam had already been installed (Figure 2-17) along with the surrounding reinforcing bars. The existing transverse reinforcing bars were orthogonal to traffic and were removed and replaced with transverse reinforcing bars that are orthogonal to the bridge skew (Figure 2-18). Due to the dimensionality of the adjacent concrete to the dam rehabilitation, this allowed for an ease of construction. The existing longitudinal reinforcing steel remained and was tied to the new transverse reinforcing bars at intersections that were not orthogonal (Figure 2-17). All existing reinforcing steel that remained were sandblasted and coated with epoxy. The construction of this bridge occured in phases, allowing one lane to be open to traffic at all times.

Table 2-13: Mixture proportions for AAA#8 used in PennDOT Project S.R. 87 in Sullivan County (aggregate weights are based on SSD)

AAA#8				
	Proportions by Volume (ft^3)	Proportions by Weight (lbs/yd^3)	PennDOT Specifications (lbs/yd^3)	Literature Recommendations (lbs/yd^3)
Cementitious Materials Content	3.5	658.0	634.5 to 752	<540
(Cement)	2.18	428.0	--	--
(GGBFS)	1.32	230.0	--	--
Water	4.33	270.0	--	--
w/cm	--	0.41	<0.43	0.42 to 0.45
Cement Paste Content (vol%)	0.35	---	0.31 to 0.43	<0.35
Coarse Aggregates AASHTO#8	9.85	1690.0	--	--
Fine Aggregates PennDOT Type A	7.70	1272.0	--	--
Design Air Content (%)	6.0	6.0	--	--
Design Slump (in)	4.0		8 Max	2.0 to 4.0
Total	27.0	3850.0	--	--

Table 2-14: Fresh concrete properties measured for District 3-0

	First Truck	Second Truck	PennDOT Specs	Literature	Comments
Measured Air Content (%)	6.0	6.2	6.0±1.5	--	OK
Measured Slump (in)	4.5	5.0	4.0±1.5	--	OK
Concrete Temperature (°F)	80	77	50 to 80	< 80	OK but at the upper limit

Figure 2-17: Final placement of bridge dam

The concrete top cover was 2.5 inches, while the bottom cover was 1.5 inches. Unlike the decks placed in District 2-0, the concrete blockouts in District 3-0 were placed to grade (Figure 2-19) and were to have their final epoxy-based surface treatment applied after milling activities. Milling would allow the deck concrete to adhere more easily to the modified concrete overlay. Also, District 3-0 uses an epoxy-based surface treatment instead of the latex-modified concrete overlay used in District 2-0.

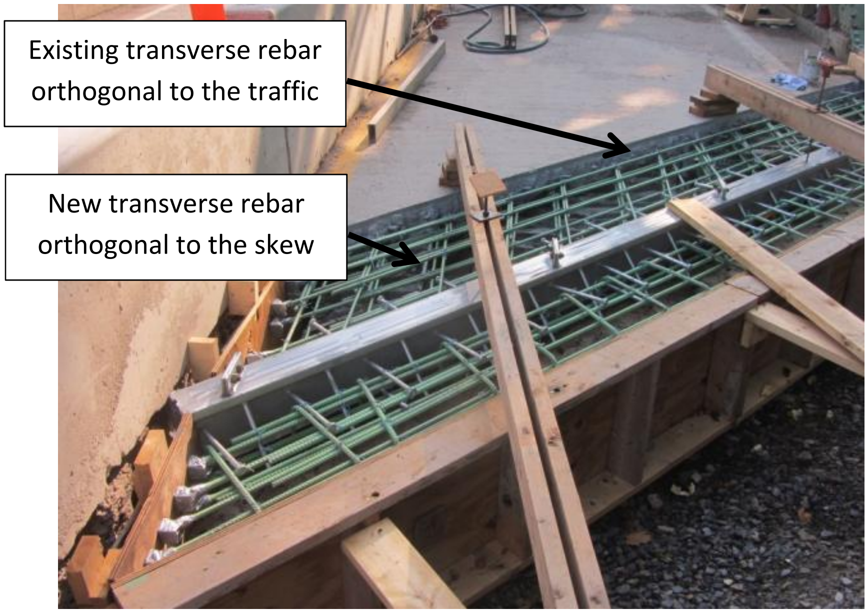

Figure 2-18: Placement of bridge with new transverse reinforcing bars orthogonal to skew

Figure 2-19: Finishing concrete to grade

The deck concrete is cured similarly to District 2-0, with multi-layer wet burlap followed by white plastic sheeting. Curing occurred within two hours of the initial concrete placement. The curing for this bridge deck occurred approximately 30 minutes after finishing activities completed, with no application of curing membranes between finishing and wet burlap application. As noted in literature, this may be too long to prevent water evaporation from the newly placed concrete. Table 2-15 provides information on the ambient conditions at the District 3-0 site. The information gathered showed the evaporation rate to be 0.08 lb/ft^2hr, which is within the acceptable range for both PennDOT specifications and literature recommendations. Similar to District 2-0, no member of the PennDOT team or the contractor team was noted as recording these values. Also, the contracting team planned to cure the concrete for only 7 days, which is less than the PennDOT specified 14 days. The contractor provided water pumps on each side of the bridge to be used to keep the burlap wet.

Table 2-15: Ambient climatic conditions for District 3-0

Ambient Conditions	Project Value	PennDOT Specs	Literature
Minimum Temperature	76 °F	--	45 °F
Maximum Temperature	78 °F	--	90 °F
Minimum Relative Humidity	84%	--	--
Maximum Relative Humidity	86%, 10.0% greater at concrete surface	--	--
Maximum Wind Speed	1.5 mph	--	--
Estimated Evaporation Rate (lbs/ft^2hr)	0.08	<0.15	0.1 to 0.2

The concrete was placed through a chute that remained between 6 inches to 2 ft from the deck at all times (Figure 2-20). The concrete was compacted using a vibrator (Figure 2-21). Vibration tended to last too long in one spot and oftentimes was used to vibrate the steel instead of the concrete. Coupling improper vibration along with a long chute placement may produce segregation of the concrete.

Figure 2-20: Placement of concrete blockout adjacent to bridge dam

Figure 2-21: Compacting concrete by vibration

2.4.5 Quality Control Results for Concrete Strength

For quality control purposes, PennDOT cast multiple 6-in by 12-in concrete cylinders according to PTM No. 611 specifications to be tested at 7, 14, and 28 days. These measurements are meant to provide a better understanding of the strength of the concrete placed in the field. PennDOT field cylinders were allowed to remain sealed inside plastic molds but at ambient conditions at the bridge location for their duration until tested. PennDOT and PSU QC cylinders were allowed to cure at ambient temperature for 7 days and then cured at room temperature until tested. The results of compressive strength are provided in Tables 2-16 and 2-17. The PennDOT QC cylinders did not pass the 1.33 strength gain requirement from 7 days to 28 days.

Table 2-16: Quality control uniaxial compressive strength results for AAA#8 concrete for S.R. 322 bridge dam rehabilitation in District 2-0

	PennDOT QC	PennDOT Field	PSU QC
7 Day (psi)	3810	---	---
14 Day (psi)	---	---	4320
28 Day (psi)	4770	5060	4730

Table 2-17: Quality control uniaxial compressive strength results for AAA#8 concrete for S.R. 87 bridge dam rehabilitation in District 3-0

	PennDOT QC	PennDOT Field	PSU QC
7 Day (psi)	---	---	---
14 Day (psi)	4620	---	5130
28 Day (psi)	5310	5540	5710

The results show the 28-day concrete uniaxial compressive strength to be between 4700 and 5710 psi. Some of these values are acceptable and not excessive in comparison with the required design strength (4500 psi). However, the concrete used on the SR 87 bridge may be deemed excessive in compressive strength.

2.5.0 SUMMARY AND CONCLUSIONS

This chapter presented the results of a review of PennDOT construction specifications, three past projects, and two current projects for rehabilitation of expansion joints/dams on concrete bridge decks, specifically:

(1) The adequacy of PennDOT concrete mix design and construction specifications related to bridge deck rehabilitation activities was evaluated based on comparison with literature recommendations. Suggestions were made to improve the specifications to reduce the risk of concrete cracking.

(2) Three past bridge dam rehabilitation projects were evaluated to ensure (a) the adequacy of reinforcing steel design against shrinkage, temperature, and mechanical service loads; and (b) the project's adherence to PennDOT specifications and literature recommendations from the standpoint of concrete material properties and construction operations.

(3) Active bridge deck rehabilitation projects were inspected to evaluate the construction activities, concrete proportions, and material properties to aid in providing suggestions to improve the durability of concrete against cracking.

The most important conclusions of this task are listed below:

2.5.1 Adequacy of PennDOT Specifications Publication 408 to Prevent Early-Age Cracking of Concrete

- The allowable cement factors in PennDOT specifications are excessive and contribute to early-age cracking of restrained concrete elements. It is recommended that PennDOT consider reducing the maximum allowable cementitious materials contents of concrete and encourage concrete suppliers to utilize methods (e.g., Shiltstone method) to improve aggregate packing and reduce the cement paste content needed to achieve proper workability. In such cases, blending of coarse aggregate gradations other than AASHTO #57, #67, or #8 should be permitted.

- The maximum allowable slump of concrete according to PennDOT specifications is 8 inches when high range water reducing admixtures are used. In contrast with the literature recommendations of 2 to 4 inches, the PennDOT slump allowance is excessive and can contribute to settlement cracking (a form of plastic shrinkage cracking).

- PennDOT specifications do not include a maximum allowable 28-day compressive strength irrespective of the strength required by the structural design. This may lead to use of excessively strong and stiff concretes by contractors, which are more prone to early-age cracking. It is recommended that PennDOT consider enforcing a maximum 28-day compressive strength of concrete at 1000 psi above the 28-day structural design compressive strength.

- PennDOT should consider reducing the allowable evaporation rate of free/bleed water from the concrete surface to 0.1 lb/ft^{2}hr. More importantly, PennDOT should strictly enforce compliance with this limit to prevent plastic shrinkage cracking of concrete. The concrete temperature, air temperature, humidity, and wind speed must be monitored throughout the placement of concrete, to allow estimation of the evaporation rate (per ACI 305R-91), and proper remediation techniques must be available for immediate application if the evaporation rate exceeds 0.1 lb/ft^{2}hr. In review of the past and active projects, it was observed that these requirements are not necessarily followed.

- Concrete surface must not be left exposed for an extended duration. The specifications must clearly mention that concrete must be protected (e.g., using fog spray) if the placement and finishing operations have to be temporarily stopped before the final finishing. Given that the current specifications prohibit the use of intermediate curing agent before final finishing, in such cases, the surface could be totally exposed without any protection against moisture evaporation. In addition, the requirement to apply the intermediate curing agent immediately after final finishing must be enforced (violations were observed during site visits). It is also recommended to require application of the first layer of pre-soaked burlap 10 minutes after the strike-off and use of a second layer within 5 minutes (McLeod et al. 2009). Continuation of water curing for 14 days is well recommended.

- Approach slabs are reinforced and are prone to a similar level of cracking risk as the bridge deck. They are, however, subject to a more relaxed standard of curing according to the current PennDOT specifications. It is recommended that PennDOT adopt similar water curing requirements for bridge approach slabs (i.e., 14 days using double layer of continuously wetted burlaps).

2.5.2 Adequacy of Reinforcing Steel Design in Past Deck Rehabilitation Projects

- It is unlikely that the cracking observed in the newly constructed concrete deck areas is a result of inadequate design of steel reinforcement. PennDOT bridge decks rehabilitated during projects 15-7PP, 180-044, and 180-058 have been properly designed with respect to the temperature and shrinkage steel requirements.

- Structural design calculations show that the support from a diaphragm next to the expansion joints must be correctly accounted for; otherwise, the reinforcing steel design of concrete deck may be inadequate based on the moment calcualtions, cracking control calculations, or both.

- This study does not recommend changes to PennDOT's Bridge Construction Specification Drawings.

2.5.3 Review of Concrete Materials and Construction Practices in the Past Deck Rehabilitation Projects

- The absence of specifying a maximum allowable compressive strength by PennDOT specifications has resulted in the use of concretes with 22% to 27% higher strength than required by design. These concretes are more prone to early-age cracking due to higher shrinkage, higher stiffness, and lower capacity for creep and stress relaxation.

- Information about concrete temperature, ambient humidity and wind speed, and estimated evaporation rate of water from the surface of fresh concrete were missing in the construction documents. The duration of water curing was also missing. PennDOT specifications require at least 14 days of water curing and also require that ambient conditions must be regularly monitored during concrete placement operations to make sure the rate of water evaporation from surface of fresh concrete never exceeds 0.15 lb/ft^{2}hr. These regulations must be strictly enforced, as they have a large impact on the plastic shrinkage cracking susceptibility of concrete.

2.5.4 Review of Concrete Materials and Construction Practices in Current Deck Rehabilitation Projects

- During visits to the active deck rehabilitation projects in District 2-0 and 3-0, it was noted that PennDOT specifications regarding prevention of plastic shrinkage cracking were not accurately followed. In particular, it was observed that the finished concrete surface remained totally exposed to evaporation for 30 to 40 minutes past final finish without application of the mandatory intermediate curing agent. In addition, PSU members did not observe the contractor or PennDOT personnel performing measurements to monitor ambient conditions (e.g., humidity and wind speed), nor did they observe available remediation equipment and procedures at the job site to make sure that the evaporation rate from the surface of concrete never exceeds 0.15 lb/ft^{2}hr.

- PennDOT specifications regarding 14-day water curing of bridge deck concrete were not accurately followed. In particular, the duration of water curing was only 7 days. Also, it was unclear how the contractor ensures that the burlap covers remain wet during the curing period. At least one observation was made by PSU team that the "wet" burlap was totally dry within 1 day after placement of concrete.

- The concrete mixture used during the District 2-0 active project had a paste content (0.37, including air) greater than the recommended paste content by the literature (0.35). This should be avoided to minimize the shrinkage and cracking of concrete.

- It was observed that the approach slab was being cured using a curing compound and not using water curing with double layer wet burlaps. This practice is allowed by PennDOT specifications, which treat the approach slab as a concrete pavement which is subject to less strict standards of curing. Given that the approach slab is reinforced and prone to early-age cracking if not sufficiently cured, the research team recommends that PennDOT modify specifications for curing of bridge approach slabs and use same requirements as used for water curing of concrete bridge decks.

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CHAPTER 3 (TASK 3)

Experimental Evaluation of the Performance and Cracking Risk for PennDOT Specified Cement Concrete Mixtures

3.1.0 INTRODUCTION

This chapter presents the results obtained during Task 3 of the project. The objectives of this task were to experimentally evaluate the quality and cracking risk of bridge deck construction and rehabilitation mixtures commonly used by PennDOT. These include concrete mixtures AAA#57, HPC#57, and AAA-P#57 (coarse aggregate #57 was used in these mixtures). This chapter provides a description of materials and test methods used as well as the results of experimental evaluations. The test data are compared with PennDOT specifications as well as recommendations from the literature review report of chapter 1. Suggestions are provided for developing modified concrete mixtures to improve the performance and mitigate early age cracking on bridge decks.

3.2.0 MATERIALS AND EXPERIMENTAL PROCEDURES

PennDOT concrete mixture AAA#57 was commonly specified on PennDOT District 3-0 bridge deck rehabilitation projects 15-7PP, 180-058, and 180-044. The exact locations of these bridges are provided in chapter 2. Due to the observed problem of early age cracking in the newly rehabilitated sections of the bridge decks, PennDOT District 3-0 subsequently implemented adjustments to the AAA#57 concrete mixture and developed new specifications for a concrete mixture named HPC#57. After the development of HPC#57, a newer concrete mixture AAA-P#57 was developed using input from the local construction industry and adopted by PennDOT. This study investigated the laboratory performance of AAA#57, HPC#57, and AAA-P#57. Given that AASHTO #57 aggregates were used as coarse aggregates in the three past projects listed above, the concrete mixtures are termed AAA#57, HPC#57, and AAA-P#57 in this study.

The mixture proportions specified by PennDOT material providers for AAA-P#57, AAA#57, and HPC#57 concrete mixtures are provided in Table 3-1. Mixture AAA#57 was used in projects 15-7PP, 180-044, and 180-058. Mixtures HPC#57 and AAA-P#57 were not implemented in these past projects. It should be noted that the cement paste content in the AAA#57 concrete is approximately 2% higher than in the other two mixtures which could result in higher susceptibility to early-age cracking, as discussed further in this chapter. To evaluate AAA#57, HPC#57, and AAA-P#57, these mixtures were duplicated in the laboratory by the Penn State research team, with proportions provided in Table 3-2. These mixtures are identical with slight adjustments per ACI 211.1 mixture proportioning guidelines due to slight differences in aggregate properties. Materials (i.e., Portland cement, blast furnace slag, coarse and fine aggregates) were procured from the same sources as those used by PennDOT contractors in the construction of past bridge deck dam projects in District 3.

The AAA#57 mixture used a water to cementitious material ratio (Portland cement plus slag) of w/cm = of 0.43, HPC#57 used a w/cm = 0.44, and AAA-P#57 used a w/cm = 0.45. Table 3-3 provides the aggregate properties. The coarse aggregate used during this study was an AASHTO grade #57 crushed limestone obtained from the Glenn O. Hawbaker quarry in Pleasant Gap, PA. Fine aggregate (sand) was in accordance with PennDOT Publication 408/2007 for a type A cement concrete sand. The fine aggregate was obtained from the Hanson Aggregate quarry in Montoursville, PA. These locations are the same locations as outlined by PennDOT concrete mixture design documents in project 15-7PP, with aggregate types being similar for the other projects.

Portland cement was procured from Essroc Cement Company in Nazareth, PA and was an ASTM C150 Type I cement. Ground granulated blast furnace slag (GGBFS) was used as a 35% replacement of Portland cement by weight for AAA#57 and HPC#57. For the AAA-P mixture, GGBFS was used as a 45% replacement of Portland cement by weight. The GGBFS procured and used throughout this work was an Essroc Cement Grade 100 GGBFS. The chemical admixtures used throughout this work were BASF brand admixtures and were obtained from a vendor in Allentown, PA. The air entraining admixture was MBVR, the plasticizing admixture was Glenium 3030, and the set retarding admixture was Pozzolith 100XR. These were proportioned properly to achieve a target fresh air content of 6.0% and slump of 4.0 inches.

Table 3-1: Mixture proportions used in PennDOT project 15-7PP (AAA#57) and PennDOT approved HPC#57 and AAA-P#57 (aggregate weights are based on SSD).

	AAA#57		HPC#57		AAA-P#57	
	Proportions by Volume (ft^3/yd^3)	Proportions by Weight ($lbs./yd^3$)	Proportions by Volume (ft^3/yd^3)	Proportions by Weight ($lbs./yd^3$)	Proportions by Volume (ft^3/yd^3)	Proportions by Weight ($lbs./yd^3$)
Cementitious Material	3.45	658.0	3.20	611.0	3.17	600
(Cement)	2.18	428	2.02	397	1.68	330
(GGBFS)	1.27	230	1.18	214	1.49	270
Water	4.54	283.0	4.31	269.0	4.33	270
Cement Paste Content	0.356	---	0.338	---	0.338	---
Coarse Aggregate	10.78	1818.0	11.02	1863.0	11.04	1860
Fine Aggregate	6.61	1078.0	6.85	1118.0	6.84	1125
Air Content	1.62	--	1.62	--	1.62	--
Total	27.00	3837.0	27.00	3861.0	27.0	3855
w/cm	--	0.43	--	0.44	--	0.45

Table 3-2: Mixture proportions for AAA#57, HPC#57, and AAA-P#57 duplicated at Penn State (aggregate weights are based on SSD)

	AAA#57		HPC#57		AAA-P#57	
	Proportions by Volume (ft^3/yd^3)	Proportions by Weight ($lbs./yd^3$)	Proportions by Volume (ft^3/yd^3)	Proportions by Weight ($lbs./yd^3$)	Proportions by Volume (ft^3/yd^3)	Proportions by Weight ($lbs./yd^3$)
Cementitious Material	3.45	658.0	3.11	611.0	3.17	600
(Cement)	2.18	428.0	2.02	397.0	1.68	330
(GGBFS)	1.27	230.0	1.18	214.0	1.49	270
Water	4.54	283.0	4.31	269.0	4.33	270
Cement Paste Content	0.353	---	0.335	---	0.338	---
Coarse Aggregate	11.02	1860	11.04	1860.0	10.73	1817
Fine Aggregate	6.46	1016	6.92	1088.0	7.15	1124
Air Content	1.62	---	1.62	---	1.62	--
Total	27.00	3817	27.00	3828	27.0	3811
w/cm	---	0.43	---	0.44	--	0.45

Table 3-3: Aggregate properties

	Used in PennDOT Past Projects	Used in PSU Mixtures
Coarse Aggregates (#57)		
Oven Dry Specific Gravity	2.71	2.70
Absorption (%)	0.59	0.23
Fine Aggregates		
Oven Dry Specific Gravity	2.63	2.52
Absorption (%)	1.31	2.02
Fineness Modulus	2.60	2.60

Concrete mixing was performed according to ASTM C 191-07 using a standard Eirich S-1 counter-current concrete pan mixer (Figure 3-1). This mixer provides full-depth and continuous shearing of the fresh concrete in order to allow for optimum mixing. Typical batches were between 1000 in^3 and 1800 in^3 (0.60 ft^3 and 1.05 ft^3).

Figure 3-2: Eirich S-1 counter-current concrete mixer

Table 3-4 provides the list of tests performed on each concrete mixture to evaluate its properties and early-age cracking tendency. The tests were performed at Penn State with the exception of the autogenous shrinkage and the restrained shrinkage (ring test), which were performed at the structural engineering laboratory of Villanova University under the supervision of Dr. Radlińska. This section provides a brief description of the specimen preparation, curing, and testing for these tests in the order presented in Table 3-4.

Table 3-4: Testing procedures performed on each concrete mixture

Test Name	Standard	Test performed at these concrete ages
Fresh Properties		
Slump	ASTM C143-05a	Fresh
Plastic air content by pressure method	ASTM C 231-10	Fresh
Mechanical Properties		
Compressive strength	ASTM C 39-05	1, 3, 7, 28, 90 days after casting
Splitting tensile strength	ASTM C 496-11	28 days
Flexural strength	ASTM C 78-10	28 days
Elastic modulus	ASTM C 469-10	1, 7, 28 days
Shrinkage and Temperature Development		
Heat of hydration	ASTM C 1064-08	Up to 200 hours
Coefficient of thermal expansion (saturated)	ASTM C 531-00	Testing commenced after 14 days of moist curing
Autogenous shrinkage	ASTM C 1698-09	Up to 28 days
Drying shrinkage (@50%RH, 74^{o}F)	ASTM C 157-08	Drying commenced at 28 days
Restrained shrinkage (Ring test)	ASTM C 1581-09	Until cracking or 28 days after casting
Other Tests		
Rapid chloride permeability	ASTM C 1202-10	28 days

3.2.1 Compressive Strength (ASTM C 39-05)

The unconfined compressive strength of concrete was measured using 4×8 inch cylinders. The cylinders were cast in three layers and compacted via 25 rods at each layer. All specimens were moist cured for the duration of their lifetime until moments before testing. The tops were smoothed with a diamond cut wet saw to provide an even surface for loading. The cylinders were tested for each mixture at 1, 3, 7, 28, and 90 days after casting. For each mixture and age, three duplicate cylinders were tested in accordance with ASTM C 39-05. The cylinders were tested using a Boart Longyear model CM-625 with a CSI Model CS-100-2A retrofit allowing for an instantaneous readout of the load imposed on the specimen, as seen in Figure 3-2. According to the standard, a rate of 35±7 psi/sec was applied to the cylinder. Both the load (lbf) and the stress (lbs./in^2) were recorded. Figure 3-3 shows a sample concrete cylinder after being loaded to failure.

Figure 3-2: Boart Longyear model CM-625 with a CSI Model CS-100-2A retrofit

3.2.2 Indirect Tensile and Flexural Strength Tests (ASTM C 490-11 and C78-10)

The indirect tensile strength testing was performed according to ASTM C 496-11. Cylinders were cast in 6×12 inch dimensions and allowed to moist cure for 28 days before testing. The cylinders were diametrically aligned in order to allow the specimen to be loaded through its centerline. The cylinders were tested using a Boart Longyear model CM-625 with a CSI Model CS-100-2A retrofit, allowing for an instantaneous readout of the load imposed on the specimen, as seen in Figure 3-2. After being properly aligned, two bearing strips (1/8-inch thick and 1-inch wide) were placed on the top and bottom of the specimen within the load cell. The specimen was then loaded according to ASTM C 496-11 standard with a rate of 11,300 lbf/min to 22,600 lbf/min. After loading the specimen to failure, the splitting tensile strength was calculated using Equation 3-1.

$$T = \frac{2P}{\pi l d} \tag{3-1}$$

where: T = the splitting tensile strength [lbs./in^2]; P = the maximum applied load indicated by the testing machine [lbf]; l = the length [in.]; d =the diameter [in.]

An average of 2 measurements performed on 2 duplicate specimens was used to determine the tensile strength of each mixture at 28 days. Figure 3-4 shows a specimen after the indirect tensile strength test was performed.

Figure 3-3: Sample concrete specimen post compressive strength testing

Flexural strength testing was performed using a four-point bending setup according to ASTM C 78-10. Concrete beams were cast with dimensions 6×6×22 inches using stainless steel molds. The beams were demolded after 48 hours and allowed to moist cure until testing at 28 days post casting. The beams were tested using a Boart Longyear model CM-625 with a CSI Model CS-100-2A retrofit allowing for an instantaneous readout of the load imposed on the specimen, as seen in Figure 3-2. Figure 3-5 provides a schematic of the testing setup. Figure 3-6 also shows the testing setup used during this study. The specimen was loaded until failure at a rate of 125-175 psi/min. Upon completion of testing, it was noted that all fracture initiations began within the middle third of the span length; therefore, the flexural strength (or modulus of rupture) can be calculated using Equation 3-2.

$$R = \frac{PL}{bd^2} \tag{3-2}$$

where: R = flexural strength [lbs./in^2]; P = maximum applied load indicated by the testing machine [lbf]; L = span length [in.]; b = average width of specimen [in.] at the fracture; d = average depth of specimen [in.] at the fracture.

Figure 3-4: Typical tensile splitting test failure (ASTM C 496-11)

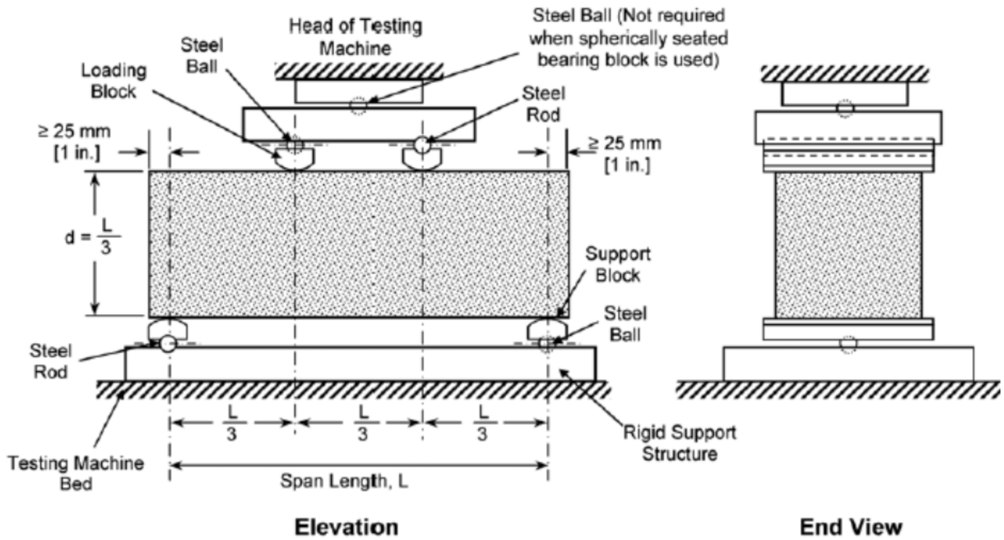


Figure 3-5: Flexural strength schematic (ASTM C 78-10)

Figure 3-6: Flexural strength test setup (ASTM C 78-10)

3.2.3 Modulus of Elasticity and Poisson's Ratio (ASTM C 469-10)

Elastic modulus and Poisson's ratio testing were performed according to ASTM C 469-10 using 4×8 inch cylinders. The cylinders were cast similar to the compressive strength cylinders, demolded at 24 hours, and moist cured until moments before testing. Testing was performed on specimens at ages 1, 7, and 28 days after casting. Each concrete cylinder was placed in a dual compressometer-extensometer connected to a data acquisition system (DAQ). They were then loaded in a Boart Longyear model CM-625 with a CSI Model CS-100-2A retrofit allowing for an instantaneous readout of the load imposed on the specimen, as seen in Figure 3-2. This setup logged the longitudinal and transverse displacement readings using two LVDTs while the specimen was loaded in compression. The load rate is similar to ASTM C 39-05 at 35±7 psi/sec. These LVDT readings were analyzed and converted to stresses and strains in order to quantify the elastic modulus and Poisson's ratio for concrete.

The static modulus of elasticity is measured through uniaxial compression testing, shown in Figure 3-7. This measurement takes into account the elastic region from the 50 millionth strain point to 40% of ultimate (failure) load for each age (Equation 3-3). This modulus is known as the chord modulus and is the most often solved for modulus of elasticity as well as the most conservative (Mindess et al. 2003). The secant modulus is solved from the beginning of the stress-strain curve to the 40% of ultimate loading point. This modulus is unreliable when compared to the chord modulus because the stress-strain origin is more unpredictable than the 50 millionth strain point (Neville 1995). Therefore, this study solves for the chord modulus and Poisson's ratio (Equation 3-4) within the same range between the 50 micro-strain point and 40% of the ultimate load. The elastic modulus should increase with concrete age while the Poisson's ratio should remain fairly constant after the first couple of days (Neville 1995).

$$E_c = \frac{S_2 - S_1}{\epsilon_2 - 0.000050} \tag{3-3}$$

where: E_c = chord modulus of elasticity [lbs./in^2]; S_2 = stress corresponding to 40% of ultimate load [lbs./in^2]; S_1 = stress corresponding to a longitudinal strain ϵ_2 50 millionths [lbs./in^2]; ϵ_2 = longitudinal strain produced by stress S_2.

$$\upsilon = \frac{\epsilon_{t_2} - \epsilon_{t_1}}{\epsilon_2 - 0.000050} \tag{3-4}$$

where: υ = Poisson's ratio; ϵ_{t_2} = transverse strain at mid-height of the specimen produced by stress S_2; ϵ_{t_1} = transverse strain at mid-height of the specimen produced by stress S_1; ϵ_2 = longitudinal strain produced by stress S_2.

It should be noted that aggregate alone and cement paste alone tend to have a linear stress-strain relationship. However, it is when the two are combined that the stress-strain relationship becomes curve-linear (Figure 3-8) (Attiogbe and Darwin 1987, Neville 1995). It is the introduction of the cement paste-aggregate interface and subsequent micro-cracking that allows a greater elastic-plastic region and provides what is truly the concrete's elastic modulus (Mindess et al. 2003). It was with this in mind that this study tests concrete (instead of mortar or paste) in order to best understand the field application of each mixture.

Figure 3-7: Setup for modulus of elasticity and Poisson's ratio (ASTM C 469-10)

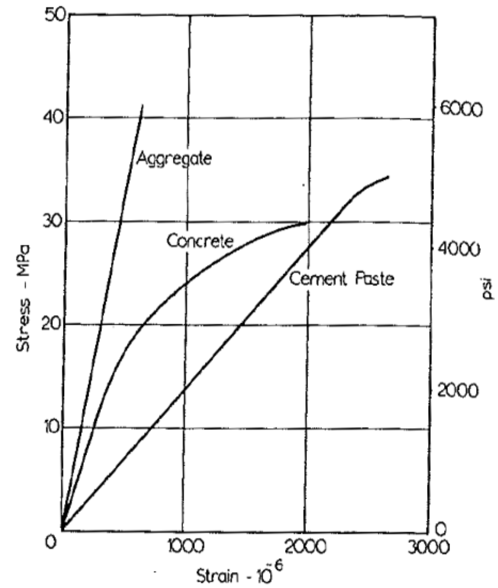


Figure 3-8: Stress-strain relationship for cement paste, aggregates, and concrete (Courtesy of Neville 1995)

3.2.4 Heat of Hydration (ASTM C 1064-08)

Testing was run according to ASTM C 1064-08. Circular holes of 6-inch diameter were cut into seven 1-inch-thick pieces of Styrofoam board. These foam boards were then stacked on top of each other to create a 7-inch-tall cylinder with a 1-inch-thick wood board underneath (Figure 3-9). After placing fresh concrete directly into the Styrofoam molds, an Omega® type T thermocouple calibrated to 32°F and 140°F was placed inside the newly formed concrete cylinder at the center of the cylinder's diameter and at mid height. The thermocouple then read the temperature once every 30 minutes and reported it to a Humboldt Model H-2680 maturity meter. Two specimens were tested for each of the AAA#57 and HPC#57 mixtures. The heat evolution of concrete allows for a better understanding of concrete's interaction with the ambient temperature and its propensity to crack (when coupled with the coefficient of thermal expansion) upon cooling from its initial peak temperature, which occurs shortly after final setting (ACI 231 2010).

Figure 3-9: Heat of hydration setup (ASTM C 1064-08)

3.2.5 Coefficient of Thermal Expansion (ASTM C 531-00)

The coefficient of thermal expansion (COTE) has a significant impact on thermal cracking tendency of restrained concrete members at early ages (ACI 231 2010). COTE quantifies the thermal strain of concrete in response to a unit increase or decrease in the temperature. As concrete hydrates, it warms due to heat of hydration. Concrete often sets near its peak temperature. As the concrete cools to ambient temperature, it undergoes thermal contraction. Thermal contractions result in tensile stresses, which increase the risk of cracking when the concrete is restrained (e.g., in bridge decks). A larger COTE results in a greater thermal contraction and a higher risk of cracking (Won 2005). The COTE of concrete is dependent on the volume fraction of cement paste (COTE typically in the range $5.5{\sim}11\times10^{-6}$ /°F), as aggregates generally show lower COTE (typically in the range $2.8{\sim}5.5\times10^{-6}$ /°F) (Emanuel and Hulsey 1977, Zoldners 1971, Meyers 1940). Large differences in coefficients of thermal expansion between the aggregate and the cement paste may cause differential expansion in the concrete and therefore cracking (Mindess et al. 2003). In addition, the moisture content of concrete can significantly affect its COTE; thermal expansion is known to be the highest in the relative humidity (RH) range of 50~70% and lower for very dry or for saturated concrete (Meyers 1940, Bažant 1970, Zoldners 1971, Sellevold and Bjøntegaard 2006). This is due to the fact that a change in temperature changes the internal RH of concrete, its moisture retention properties, as well as the surface tension of water. As such, the temperature change can cause hygrothermal shrinkage or swelling due to moisture loss or gain, and this is in addition to any volume changes due to thermal expansion or contraction of the solid skeleton (Grasley 2003). Along with the moisture/RH, lower porosity of cement paste (achieved by lower w/cm and age) could reduce its COTE (Emanuel and Hulsey 1977).

In this work, the COTE was found for mortar mixtures at the saturated condition. Mortar bars (1×1×10 inches according to ASTM C 490-11) were used by excluding coarse aggregates from mixture proportions provided in Tables 3-1 and 3-2. This is specified by the ASTM C 531-00 standard to limit the temperature gradients that could develop in larger prisms containing coarse aggregate. Mortars were mixed according to ASTM C 305 and cast in prism molds using a vibrating table. Embedded nickel studs were used to facilitate length measurements. The studs had a coefficient of thermal expansion of 7.2×10^{-06}/°F, which was accounted for in the COTE calculations (Equation 3-5). Testing began after the specimens had been moist cured for 14 days. The results from four duplicate prisms were used and averaged to determine the COTE of each mixture in the saturated condition. The saturated specimens were heated from room temperature (~73.5°F) to a temperature of 176°F while fully submerged in saturated limewater. After at least 16 hours at 176°F, the specimens' length was recorded using a Humboldt digital comparator model BG2600-16001 (Figure 3-10). The temperature of the limewater bath was checked periodically using a thermometer. Mortar bars were removed from the 176°F bath one by one and their length measured to the nearest 0.0001 inch. The specimens were then submerged back into the limewater bath and cooled to a temperature of 140°F. After at least 16 hours at 140°F, the specimens' length was recorded. This temperature cycle (140°F to 176°F and reverse) continued until specimens reached constant lengths at both 140°F and 176°F.

$$COTE = \frac{Z - Y - W}{T(W - X)} \tag{3-5}$$

where: Z = length of mortar bar, including studs, at elevated temperature [in.]; Y = length of stud expansion [in.]= $X * T * k$, where k is the linear coefficient of thermal expansion per °F of the studs; W = length of bar, including studs, at lower temperature [in.]; T = temperature change [°F]; and X = length of the two studs at lower temperature [in.].

Figure 3-10: Humboldt digital comparator model BG2600-16001 (ASTM C 531-00 setup)

3.2.6 Autogenous Shrinkage (ASTM C 1698-09)

Autogenous shrinkage of concrete is the shrinkage that occurs in the absence of external drying (e.g., in sealed specimens) and due to self-desiccation of concrete during hydration of Portland cement (please see section 1.1 of Task 1 report for further description). To evaluate the magnitude of autogenous shrinkage in the AAA#57 and HPC#57 mixtures, four cement paste specimens per mixture were prepared and the autogenous shrinkage strain development over time was recorded. The linear autogenous deformation of the paste specimens was measured in a sealed condition using the corrugated tube protocol (Jensen and Hansen 1995, Sant et al. 2006, Radlińska et al. 2008). The corrugated tube protocol involves the encapsulation of fresh cement paste (approximately 30 minutes after water is added to the mixture) in a corrugated polyethylene tube (Figure 3-11). The tube has a length-to-diameter ratio of 400 mm to 30 mm and a significantly greater stiffness in the radial direction than the longitudinal direction. This allows transformation of the volumetric deformation into longitudinal deformation (Radlińska et al. 2008). All specimens were maintained in a 73°F environment and their shrinkage was measured using a digital dilatometer. Figure 3-12 shows the setup similar to the one used in this study. The length measurements were recorded at 3 and 18 hours, then once a day through 3 days and once every 7 days afterwards through 28 days.

It is understood that the w/cm will have the greatest impact on autogenous shrinkage. Typically all concrete with a w/cm below 0.42 will experience autogenous shrinkage; however, pastes having a w/cm greater than 0.42 may also exhibit a volume change due to autogenous shrinkage (Bentz et al. 2001, Baroghel-Bouny 1996). It is noted that the autogenous shrinkage phenomenon occurs almost exclusively in sealed/closed systems where there is limited or no access to external curing water (Radlińska et al. 2008). Also, in most field applications and laboratory tests, the concrete is typically concealed for the first 24+ hours of its lifetime. Therefore, autogenous shrinkage will be examined with respect to shrinkage occurring during the first 24 hours (or the concealed time). Although studies have shown autogenous shrinkage occurring later in the cement paste's lifetime (i.e., 60~100 days), it is quite rare for concrete to be concealed for this length of time (ACI 231 2010) and this was not considered in this study. Jensen and Hansen (1996) showed that pozzolans (fly ash, GGBFS, and silica fume) can increase the autogenous volume change of cement pastes. Although cement pastes routinely exhibit a greater shrinkage than cement mortars or concretes (Holt 2002, Tazawa et al. 1995); cement paste was tested in the study considering the corrugated tubing size restriction. Pickett's Equation (Equation 3-6) was used in order to relate the autogenous shrinkage of cement paste into shrinkage of concrete (1956).

$$\varepsilon_{con} = \varepsilon_{paste}\left(1 - V_{agg}\right)^{n} \tag{3-6}$$

where: ε_{con} = shrinkage of concrete, [µm/m]; ε_{paste} = shrinkage of cement paste, [µm/m]; V_{agg} = aggregate volume fraction; n = aggregate parameter depending on aggregate stiffness (assumed to be 1.2).

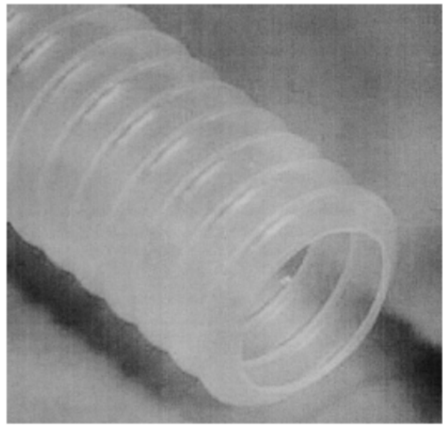


Figure 3-11: Corrugated tube used for autogenous shrinkage testing during this study

(Courtesy of Jensen and Hansen 2001)

Figure 3-12: Experimental setup to measure the autogenous shrinkage according to

ASTM C 1698-09

3.2.7 Drying Shrinkage (ASTM C 157-08)

Concrete specimens (Figure 3-13) were cast in 3×3×11 inch rectangular molds with embedded nickel alloy studs. Three duplicate specimens were tested for each of the AAA#57 and HPC#57 mixtures. The initial comparator measurements were made using a Humboldt digital comparator model BG2600-16001 upon demolding the specimens 24 hours after casting. After initial measurements, the specimens were submerged in a limewater bath for 27 days. After a 27-day submersion, the specimens' length was measured and drying commenced. The specimens were allowed to dry in an ambient environment of 73±2°F and 50±5% RH. Comparator and weight measurements were then recorded with final measurements occurring 157 days after casting (i.e., total drying time was up to 129 days).

Figure 3-13: ASTM C 157-08 setup for drying shrinkage of concrete

Drying shrinkage represents the strain caused by the loss of water from hardened concrete (Mindess et al. 2003). The effects of drying shrinkage in concrete can lead to cracking or warping of structures due to external or internal restraints. This can be seen clearly on concrete pavements or slabs if contraction joints are not properly placed. Jointing will occur in order to prevent irregular, random cracking and focus it on a particular location in order to be sealed at a later date (Mindess et al. 2003). Although shrinkage is a paste property phenomenon (Radlińska et al. 2008), concrete specimens were examined in order to better understand the field applicability of each mixture. The mechanisms affecting the volume change of concrete (bulk shrinkage) are capillary stresses, disjoining pressures, and changes in surface free energy (Mindess et al. 2003). Those three mechanisms dominate the bulk shrinkage within the typical field temperatures (15~95°F) and relative humidity (40% to 100%) (Figures 3-14a and 3-14b) (Mindess et al. 2003, Radlińska et al. 2008). It may be noted that plastic shrinkage cracking (moisture lost before concrete has set) may contribute more to this study's cracking issue than drying of hardened concrete.

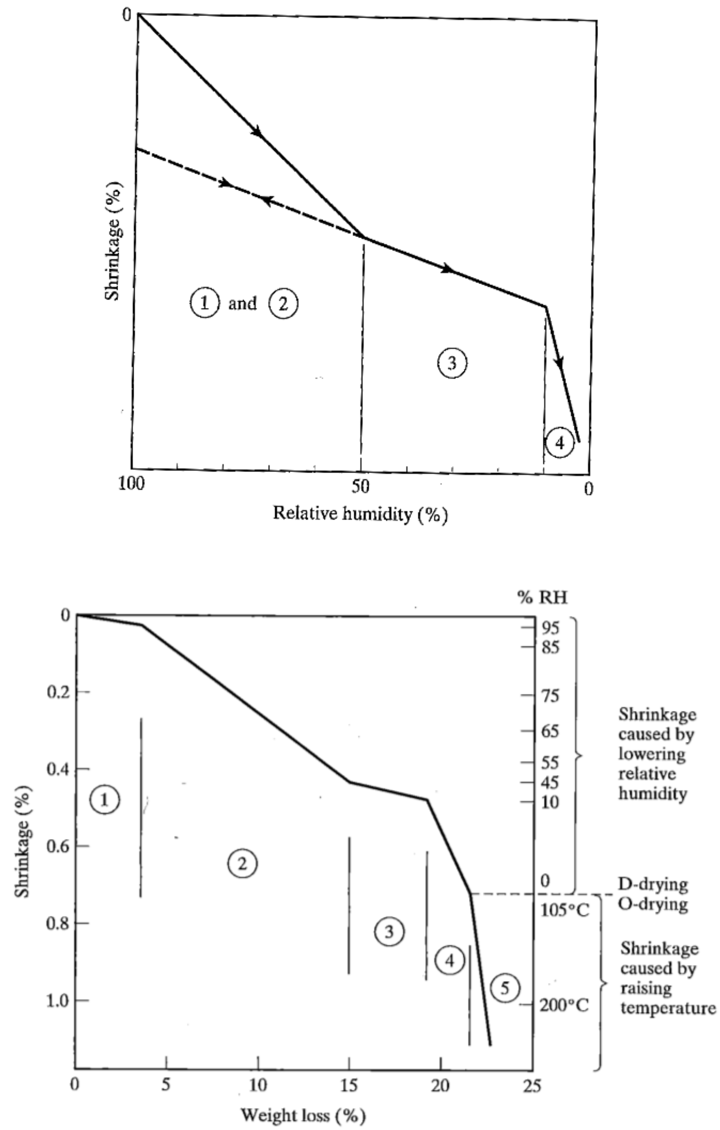


Figure 3-14: Relationship between ambient relative humidity (%RH) and (a) weight loss, and (b) drying shrinkage of concrete (Mindess et al. 2003)

The results of drying shrinkage measurements should also accounts for the autogenous shrinkage development during the first 24 hours. Since the first measurement of drying shrinkage (according to ASTM C 157-08) occurs after 1 day, the autogenous shrinkage that has occurred during the first 24 hours (or the concealed time) is not taken into account. Combining the two measurements will provide a more realistic interpretation of the total shrinkage occurring in the system (Sant et al. 2006). This combination however was not considered during this study, since drying commenced after 27 days of limewater saturation.

3.2.8 Restrained Shrinkage; Ring Test (ASTM C 1581-09)

In addition to the free autogenous and drying shrinkage measurements, the ASTM C 1581-09 restrained ring test was performed on AAA#57 and HPC#57 concrete mixtures in order to evaluate the cracking susceptibility of these two analyzed concrete mixtures. In this test setup, schematically shown in Figure 3-15, a concrete annulus was cast around a steel ring. After 24 hours of curing under wet burlap in laboratory-controlled conditions, the specimen was demolded (Figure 3-16a), the top surface was sealed with aluminum tape (Figure 3-16b), and the concrete was allowed to dry from its outside circumference (ASTM C1581-09) in environmentally controlled conditions of 73°F and 50% RH. As the concrete ring was allowed to dry, shrinkage took place due to drying, self-desiccation, and heat loss that resulted in a radial pressure applied to the steel ring. This pressure leads to the development of tangential tensile stresses in the concrete. The resulting deformation was measured by four symmetrically placed strain gages, mounted on the inner surface of the steel ring (mid-height). As the stresses inside the concrete grow with time, they might eventually reach the tensile strength of the material, leading to concrete ring cracking. The time of cracking is indicated in the data set as a sudden drop of measured strains on the steel ring. The age at which cracking occurs and the stress magnitude at the time of cracking provide a good indication of the susceptibility of the concrete mixture to early-age cracking.

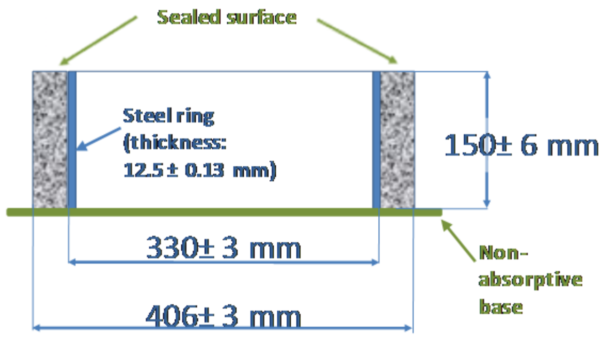


Figure 3-15: Geometry of the ring specimen per ASTM C 1581-09

(a) (b)

Figure 3-16: Ring specimens during casting: (a) the ring specimen right after demolding; (b) top surface of the concrete ring sealed with aluminum tape

3.2.9 Rapid Chloride Permeability (ASTM C 1202-10)

The ability of concrete to resist the penetration of aggressive elements (e.g., chloride ions) is key to the durability of reinforced concrete structures. External chloride ions (e.g., due to application of deicing salts or in marine environments) penetrate through concrete's cover layer and cause corrosion of the reinforcing steel bars (Figure 3-17) (Berke et al. 1988). The steel corrosion products (rust) have a much larger volume, up to 7 times the volume of the original steel. This volume expansion causes large tensile stress development inside concrete and results in cracking and spalling of the concrete's cover, which exposes the corroding rebar (Berke et al. 1988).

To evaluate the resistance of concrete mixtures against penetration of chloride ions, ASTM C 1202-10 was performed. 4×8 inch concrete cylindrical specimens were prepared and moist cured for 24 days. This test requires a considerable amount of preparation time; therefore, in order to test the specimens at 28 days, preparation began at 24 days. After curing, the specimens were cut into 2-inch thick disks from the center of the specimen. Then, epoxy resin was applied to the exterior sides of the cylinder to prevent lateral moisture loss. After the epoxy resin dried, the concrete specimens were subjected to a vacuumed drying inside a desiccator for 3 hours. After 3 hours, the desiccator was partially filled with de-aired (boiled) water to submerge all specimens. The vacuum was allowed to run for an additional 1 hour. Next, the vacuum was shut off and the specimens were allowed to soak for an additional 18 hours. This procedure is intended to fully saturate the concrete pores with water. The specimens were subsequently removed and loaded into two half-cells made of Plexiglas and sealed via silicone rubber (Figure 3-18). Each half-cell had a reservoir that was filled with a solution of either 3.0% NaCl or 0.3N NaOH. After the silicone was allowed to cure overnight, thus concluding the 4-day preparation process, the cells were filled with the aforementioned solutions and subjected to a 60V DC voltage across the specimens' thickness. The voltage was applied for 6 hours with the measurements recorded automatically every 30 minutes by the RLC instrument model 164A Test Set Power Supply. Two specimens of each mixture were tested at an age of 28 days.

Table 3-5: Qualitative description of concrete chloride ion penetrability per ASTM C 1202-10

Charge Passed (coulombs)	Chloride Ion Penetrability
>4000	High
2000-4000	Moderate
1000-2000	Low
100-1000	Very Low
<100	Negligible

Figure 3-17: Corrosion of steel reinforcing bars in concrete (Courtesy of Matco Services, Inc.).

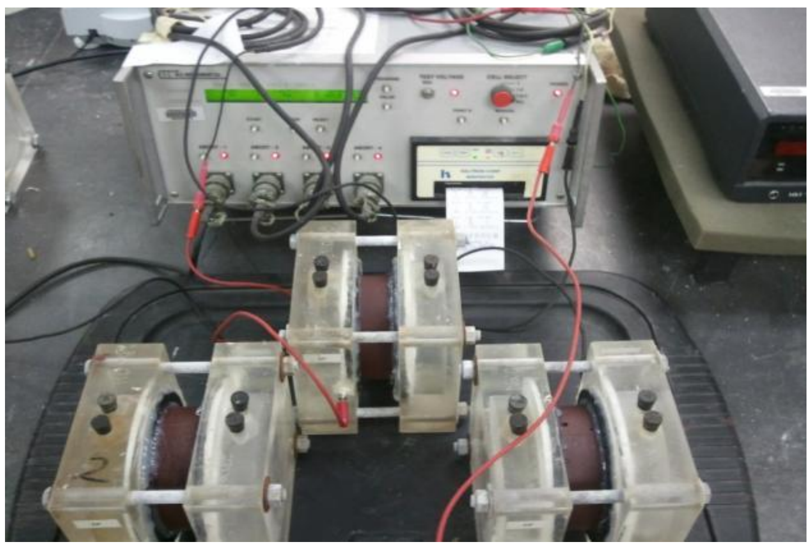


Figure 3-18: The rapid chloride permeability test setup (ASTM C 1202-10)

Concrete with a higher w/cm has a greater volume of capillary porosity and thus allows for easier penetration of moisture and ions. ASTM C 1202-10 provides a qualitative evaluation of concrete resistance to ion penetration using the magnitude of the electrical charge passed during the 6-hour test (Table 3-5). Large electrical current passing through high w/cm specimens can produce heat, thus increasing the temperature of the specimen (Stanish et al. 1997). The increase in temperature serves as a positive feedback, leading to an artificial increase in the electrical current and the charge passed (Mindess et al. 2003). For this reason, during the test, the temperatures of the solutions were monitored periodically using a thermometer to ensure that they do not exceed 190^{o}F. In addition, when the current exceeded 300 mA at the test's conclusion, the RCPT values were corrected to account for overheating. The correction was based on the current passed during the first 30 minutes of testing (Mindess et al. 2003). The new ASTM C1760 test method allows for the use of electrical current passing during the first 1 minute of the test.

3.3.0 RESULTS AND DISCUSSION

The following sections present the full testing results for AAA#57, HPC#57, and AAA-P#57 concrete mixtures.

3.3.1 Fresh Properties

The design slump of the three mixtures was 4.0 inches and measurements in the range 4.0±1.5 inches are considered acceptable according to PennDOT specifications. The slump was attained via the use of a water reducing admixture. Between 7.0 and 8.5 fl. oz. of the admixture per a unit weight of cementitious material were used to achieve measured slumps between 3.5 and 4.5 inches. According to PennDOT specifications, plastic air content in the range 6.0±1.5% is required. No air entrainment was necessary to attain the design fresh air content of 6.0%. Air content ranged between 5.4% and 6.4% when measured per ASTM C 231. The HPC#57 and AAA-P#57 utilized a set retarding admixture at a dosage of 2.7 fl.oz. per unit weight of cementitious materials, according to PennDOT documents.

3.3.2 Mechanical Properties

The mechanical testing included the compressive strength test, indirect tensile strength test, the flexural strength test, and measurements of the static (chord) elastic modulus and Poisson's ratio of concrete in compression. Table 3-6 provides the results of the compressive strength test as a function of time for the three mixtures. Each data point is the average of strength measurements from 3 duplicate specimens. Table 3-7 shows the 7- and 28-day strength measurements of AAA#57 concrete cylinders cast during past PennDOT projects. The 28-day structural design strength and the minimum allowable compressive strengths are also included. This information was extracted from PennDOT construction documents.

Table 3-6: Compressive strength, elastic modulus, and Poisson's ratio of AAA#57, HPC#57, and AAA-P#57 mixtures prepared at PSU

	AAA#57			HPC#57			AAA-P#57		
DAY	Strength (lbs./in^2)	Elastic modulus (lbs./in^2)	Poisson's ratio	Strength (lbs./in^2)	Elastic modulus (lbs./in^2)	Poisson's ratio	Strength (lbs./in^2)	Elastic modulus (lbs./in^2)	Poisson's ratio
1	1580	2.91×10^6	0.05	1600	2.54×10^6	0.09	820	2.12×10^6	0.03
3	3070	---	---	3090	---	---	2440	---	---
7	4160	4.61×10^6	0.12	3780	4.67×10^6	0.12	3900	4.09×10^6	0.10
28	6100	5.35×10^6	0.13	5550	5.35×10^6	0.11	5120	5.34×10^6	0.11
90	6250	---	---	5930	---	---	5700	---	---

Table 3-7: PennDOT required 28-day strength as well as measured strength from cylinders cast during three past PennDOT projects

	15-7PP (PennDOT)	180-044 (PennDOT)	180-058 (PennDOT)
Lab Tested 7 day	3705	3795	4165
Lab Tested 28 day	5527	5510	5721
Design 28 day Structural	4000	4000	4000
Design 28 day Minimum	4500	4500	4500

As can be seen in Table 3-7, all PennDOT implemented mixtures as well as duplicate PSU mixtures for AAA#57, HPC#57, and AAA-P#57 show greater strength values than the required minimum 28-day strength (i.e., 4500 psi). The measured 28-day cylinder strengths exceed the minimum strength requirements by 14% to 36%. The measured values are also greater than the literature recommended 28-day strength values of 3000 to 4500 psi for bridge deck applications (Krauss and Rogalla 1996). This can be attributed to the low w/cm used by PennDOT in field applications. The w/cm used during this study was 0.43~0.45 which is lower than literature recommended 0.45~0.53 (Krauss and Rogalla 1996). As noted in chapter 1 of this report, strengths higher than specified by structural design are not required and can exacerbate deck cracking (Frosch et al. 2003). This is due to the fact that higher compressive strengths are usually achieved by increasing cement content and reducing w/cm. This will result in a higher heat of hydration, higher paste contents, and higher autogenous and drying shrinkage. A higher modulus of elasticity and lower creep will occur as well. All of these conditions favor higher stress development and higher cracking risk for the concrete bridge deck. Overall, the use of excessively strong concretes should be avoided.

On the other hand, due to Pennsylvania's climate and the need to protect against a severe freezing and thawing condition as well as corrosion induced by deicing salts, it would not be advisable to increase the w/cm to better match strength requirements. However, the cement paste content can be minimized to the extent that workability requirements can still be met. Also, the dosage of slag (GGBFS) can be increased (i.e., higher amounts of Portland cement can be replaced) to slightly reduce the early-age strength (seen for the AAA-P#57 mixture with 45% cement replacement by weight with GGBFS) and elastic modulus with the added benefit of reducing the heat of hydration of concrete and improving the long-term strength and durability. High slag mixtures should be used with caution in colder months due to reduced heat of hydration and slow strength development due to low concrete temperature.

Table 3-6 also provides the results of the modulus of elasticity and Poisson's ratio at 1, 7, and 28 days for the mixtures. The elastic (chord) moduli reported in Table 3-6 follow the expected trend of increasing as the concrete ages and are typical for concrete's testing at given ages (Neville 1995, Mindess et al. 2003). In the absence of experimental data, ACI 318 (2011) recommends estimating the secant modulus of elasticity using Equation 3-7 based on the compressive strength of concrete. As discussed in Section 3.2.3, the secant and chord moduli are quite similar (may differ by less that 2%); therefore, the calculated secant moduli may be used in a comparison against measured chord moduli.

$$E_c = 57{,}000\sqrt{f'c} \tag{3-7}$$

where: E_c = elastic modulus of concrete [lbs./in^2]; $f'c$ = unconfined compressive strength of concrete [lbs./in^2].

In comparison with measured elastic moduli included in Table 3-6, the ACI formula underestimates the modulus by approximately 19%. It is noted that concrete with a greater chord/secant modulus has a greater propensity to cracking due to a higher elastic stress corresponding with similar shrinkage strains, as well as lower creep and stress relaxation (Neville 1995). The Poisson's ratio remains consistent from 7 days to 28 days. It should be noted that the Poisson's ratio reported in this study may be slightly lower than typical concrete values reported in the literature (Neville 1995, Mindess et al. 2003).

The results of the indirect tensile strength and the flexural strength tests are provided in Table 3-8. Indirect tensile strength measurements are typically between 8% and 14% of the compressive strength (Mindess et al. 2003). AAA#57 and HPC#57 indirect tensile strengths are slightly lower than estimated values from the literature, while the AAA-P#57 mixture is within the literature range. Flexural strengths are typically between 11% and 23% of the compressive strength (Mindess et al. 2003). AAA#57, HPC#57, and AAA-P#57 flexural strengths are typical according to the literature. In order to better understand the risk of cracking, the flexural and tensile strength of the concrete mixtures were interpolated in order to show values at 1 and 7 days. Equations 3-8 and 3-9 show these interpolations, respectively, as instructed from literature (Mindess et al. 2003). This is discussed further in Section 3.3.4.

$$f'r = 2.80f'c^{\frac{2}{3}} \tag{3-8}$$

$$f't = 4.34f'c^{0.55} \tag{3-9}$$

where: $f'r$ = concrete modulus of rupture at a time t [lbs./in^2]; $f't$ = concrete tensile strength at a time t [lbs./in^2]; $f'c$ = concrete compressive strength at time t [lbs./in^2].

Table 3-8: Indirect tensile and flexural strengths of AAA#57 and HPC#57 mixtures

	AAA#57		HPC#57		AAA-P#57	
DAY	Indirect tensile strength (lbs./in^2)	Flexural strength (lbs./in^2)	Indirect tensile strength (lbs./in^2)	Flexural strength (lbs./in^2)	Indirect tensile strength (lbs./in^2)	Flexural strength (lbs./in^2)
28	375.0	934.5	409.0	852.5	448.0	954.0

3.3.3 Shrinkage and Temperature Development

3.3.3.1 Heat of hydration and coefficient of thermal expansion

The results of heat of hydration monitoring for AAA#57, HPC#57, and AAA-P#57 mixtures are provided in Figures 3-19, 3-20, and 3-21, respectively. Both the AAA#57 and HPC#57 show similar heat signatures, with peak temperatures of up to 97°F. The AAA-P#57 mixture shows a heat signature lower than 95°F. However, the HPC#57 and AAA-P#57 mixtures utilize a set-retarding admixture that slightly delayed its heat evolution. This peak temperature is reasonable for Type I cement with 35% GGBFS replacement by weight (Mindess et al. 2003). AAA-P#57 concrete used 45% GGBFS by weight and therefore it is reasonable that it would provide a lower peak temperature. PennDOT specifications discuss that the concrete temperature at placement of bridge decks shall not exceed 80°F; however, the specifications do not discuss the early age heat evolution.

The results of the mortar coefficient of thermal expansion measurements for the AAA#57, HPC#57, and AAA-P#57 mixtures are provided in Table 3-9. HPC#57 and AAA-P#57 have a slightly lower COTE, which is reasonable due to their lower paste content. These values are reasonable if not slightly lower than those reported in the literature (Mindess et al. 2003, Meyers 1940, Neville 1995, Chern and Chan 1989). Based on simple mixture rules (Mindess et al. 2003), concrete with 40% limestone coarse aggregate by volume can be interpolated using limestone's COTE and mortar's COTE. Considering 40% of the concrete's volume consists of limestone (COTE equals 3.33×10^{-06}/°F) and the remaining 60% of concrete's volume consists of mortar (COTE equals values measured in Table 3-9), the concrete's calculated COTE can be presented in Table 3-9.

Thermal strains can be estimated based on COTE and heat of hydration peak temperatures (97°F or 95°F) shown in Figures 3-19, 3-20, and 3-21, which must cool down to ambient temperature of 73°F. For the AAA#57 mixture with COTE = 4.83×10^{-06}/°F, this results in a thermal contraction strain of approximately 116 με. For the HPC#57 mixture with COTE = 4.69×10^{-06}/°F, this results in a thermal contraction strain of approximately 113 με. For the AAA-P#57 mixture with a COTE = 4.66×10^{-06}/°F, this results in a thermal contraction strain of approximately

112 με. These values should be added to drying shrinkage strains when calculating the mixture's risk of cracking (section 3.4).

Table 3-9: COTE results for AAA#57 and HPC#57 saturated mortars and interpolated concretes

Mixture	COTE (strain/°F)
Mortar AAA#57 (/°F) (measured)	5.84×10^{-06}
Mortar HPC#57 (/°F) (measured)	5.62×10^{-06}
Mortar AAA-P#57 (/°F) (measured)	5.60×10^{-06}
Concrete AAA#57 (/°F) (estimated)	4.83×10^{-06}
Concrete HPC#57 (/°F) (estimated)	4.69×10^{-06}
Concrete AAA-P#57 (/°F) (estimated)	4.66×10^{-06}

3.3.3.2 Autogenous and drying shrinkage

The average results of autogenous shrinkage measurements of the AAA, HPC, and AAA-P cement paste mixtures are provided in Figure 3-22. The variability observed in the test results is inherent to the experiment; ASTM C1698 reports a single operator standard deviation of 28 με for similar w/cm cement pastes. The results show an average autogenous strain for the first 80 hours (from cement-water contact) to be approximately 15 με for the AAA cement paste, approximately 12 με for the HPC, and approximately -26 με for AAA-P. Overall, these values are negligible, which is due to the w/cm of the paste being higher than the 0.42 value required for significant autogenous shrinkage development (Bentz et al. 2001). Appendix B shows the autogenous strain development for each paste specimen over the 28-day time span. It should be noted that the autogenous strain starts to grow beyond 80 hours and reach considerable but not high values at 28 days. This may be due to gradual reaction of GGBFS, as suggested by Tazawa and Miyazawa (1995). Also, concrete containing GGBFS develops smaller pores over time, which increases the magnitude of capillary pressure, resulting in autogenous deformations (Lura et al. 2001, Lee et al. 2006). Such strains, however, would only develop if cement paste remains sealed for 28 days, which is unlikely in field conditions where concrete is externally moist cured and subsequently exposed to ambient conditions (e.g., rain). It should also be noted that these measurements correspond to the calculated autogenous shrinkage of the concrete, which is determined using Pickett's Equation (Equation 3-6) for the given aggregate volume fraction (64.5% for AAA#57 and 66.5% for HPC#57 and AAA-P#57).

Figures 3-23 and 3-24 show the drying shrinkage and drying mass loss of AAA#57 and HPC#57 specimens. The drying period started after 28 days of moist curing (i.e., demolding at 1 day and submersion in limewater bath for 27 days) and continued until 157 days after casting (104 days for AAA-P#57). Each data point in Figures 3-23 and 3-24 is the average of measurements performed on three duplicate prism concrete specimens. The initial gain in mass and length (expansion) are due to absorption of water and swelling during the limewater bath submersion. It is the subsequent drying of this water which results in drying shrinkage. Both shrinkage and mass loss grow rapidly soon after drying initiation; however, they start to plateau after approximately 100 days, showing the beginning of equilibrium with the ambient (73±2°F and 50±5% RH). It should however be noted that the AAA-P#57 concrete mixture has not reached equilibrium after 76 days of drying (104 days since casting). The ultimate shrinkage of AAA#57 is recorded as 138 + 276 = 414 $\mu\varepsilon$ and that of HPC#57 is 78 + 299 = 377 $\mu\varepsilon$ at 157 days since casting (i.e., 129 days of drying). The drying shrinkage for the AAA-P#57 concrete at 104 days since casting is 147 + 316 = 463 $\mu\varepsilon$, and continues to gradually increase with time. These results indicate that the drying shrinkage of AAA-P#57 mixture is larger than the AAA mixtures by at least 11.8%. The drying shrinkage of AAA-P#57 mixture is larger than the AAA mixtures by at least 22.8%. These larger shrinkage values are likely to be caused by a higher dosage of GGBFS as suggested by the literature.

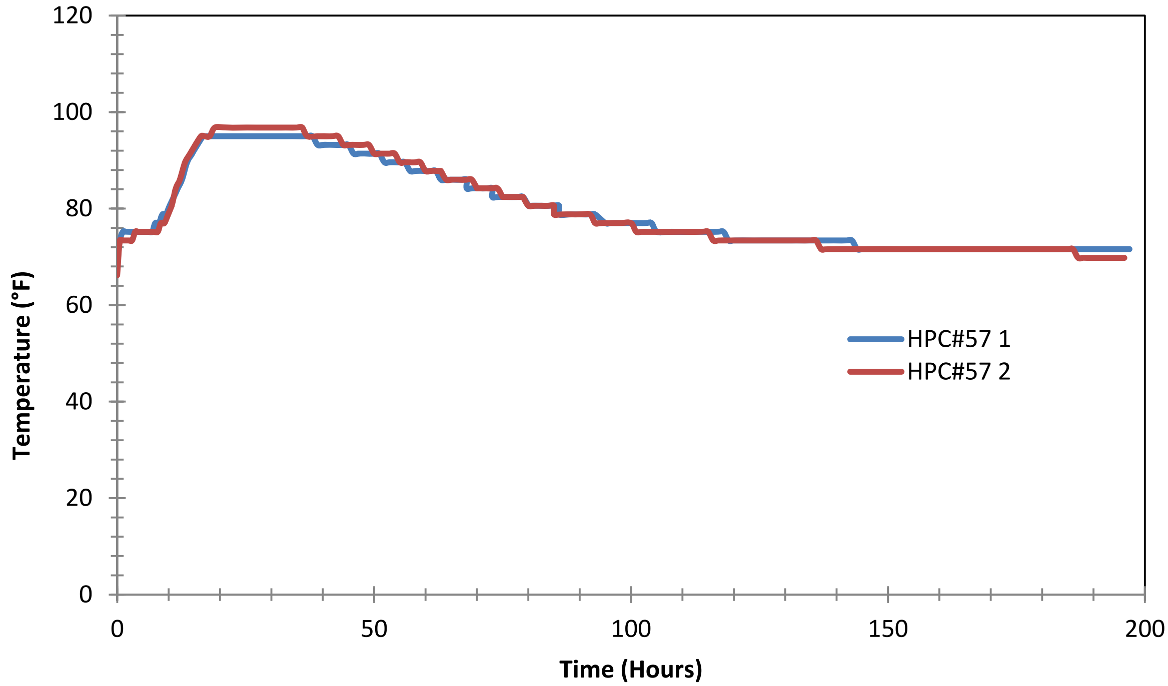


Figure 3-19: HPC#57 heat of hydration evolution

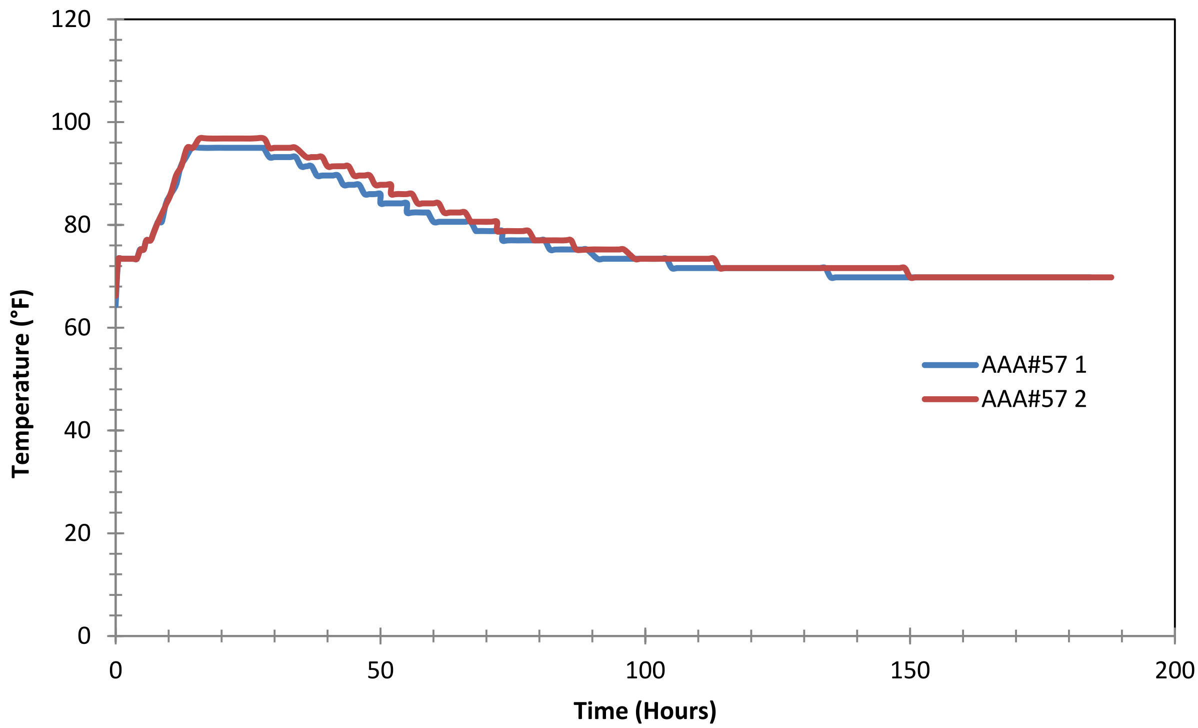


Figure 3-20: AAA#57 heat of hydration evolution

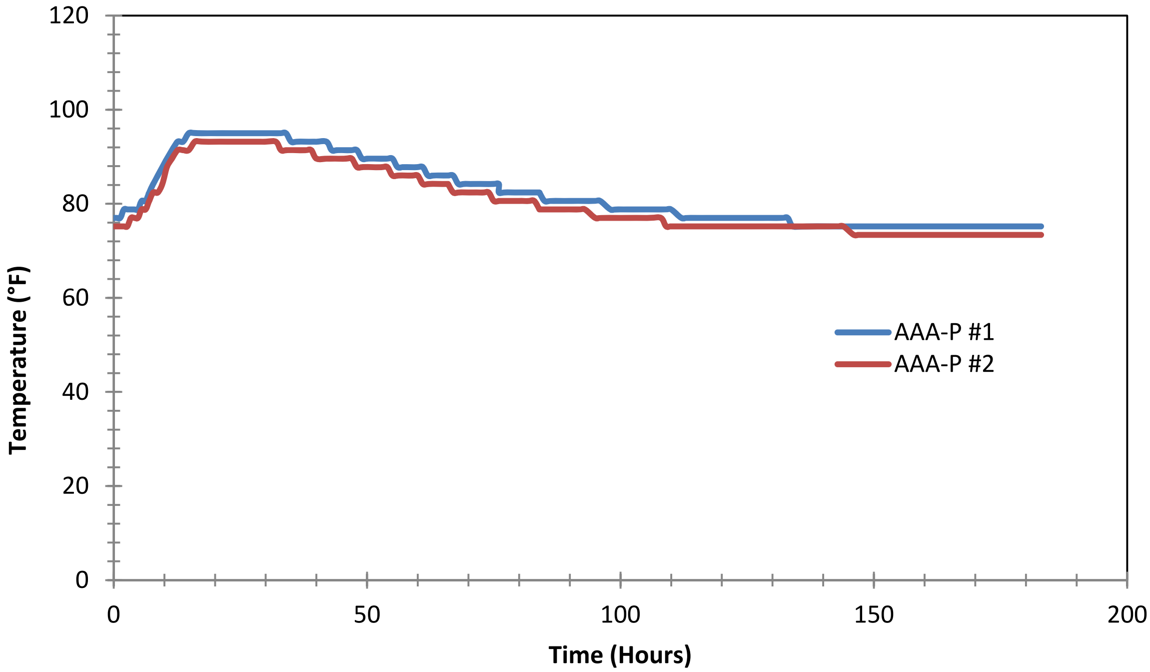


Figure 3-21: AAA-P#57 heat of hydration evolution

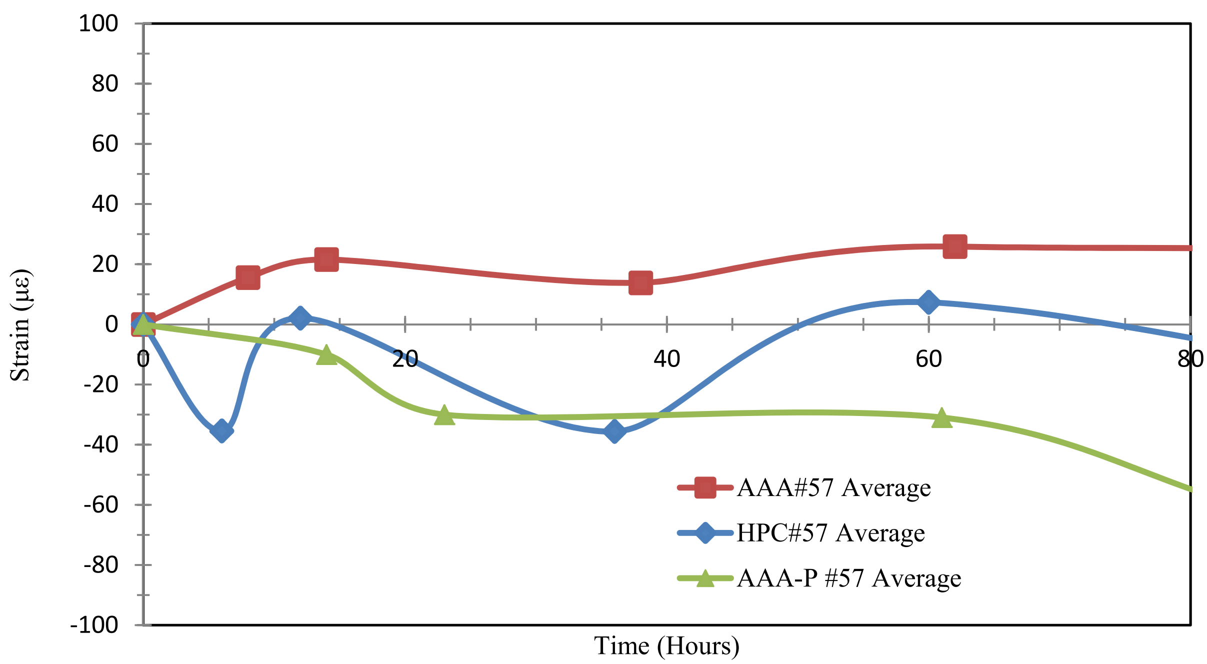


Figure 3-22: Average autogenous shrinkage development over the first 80 hours

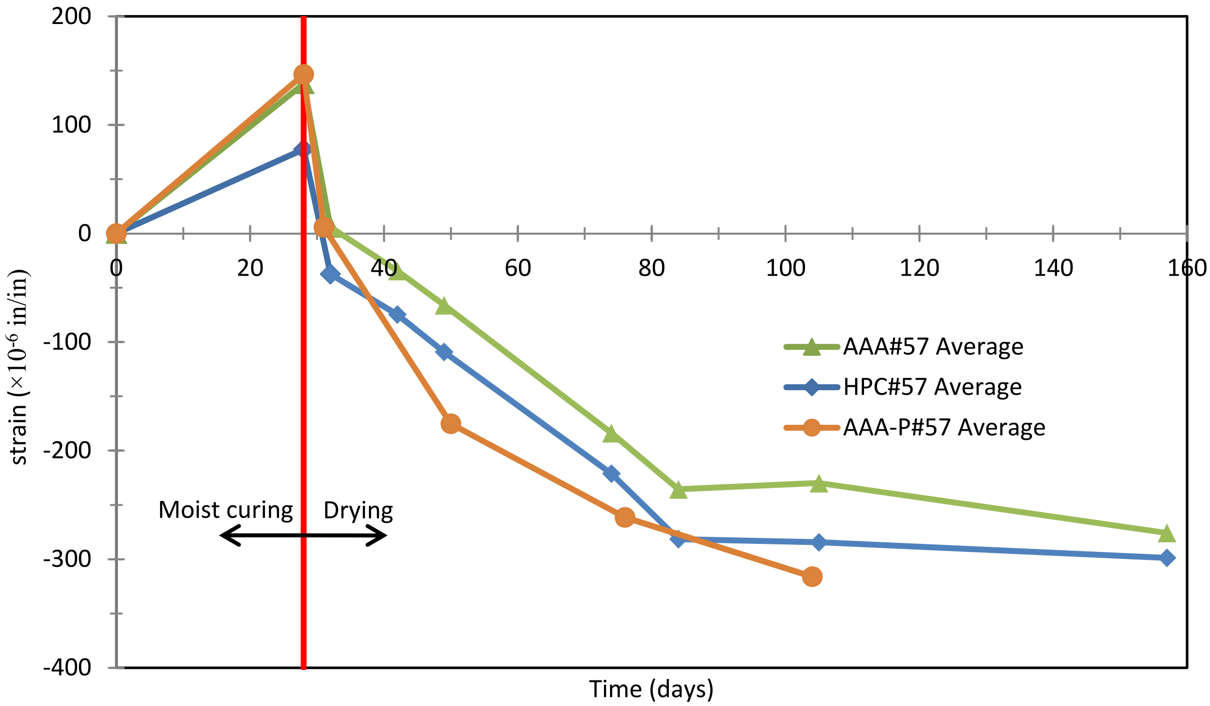


Figure 3-23: Drying shrinkage strain development over time for AAA#57, HPC#57, and AAA-P#57

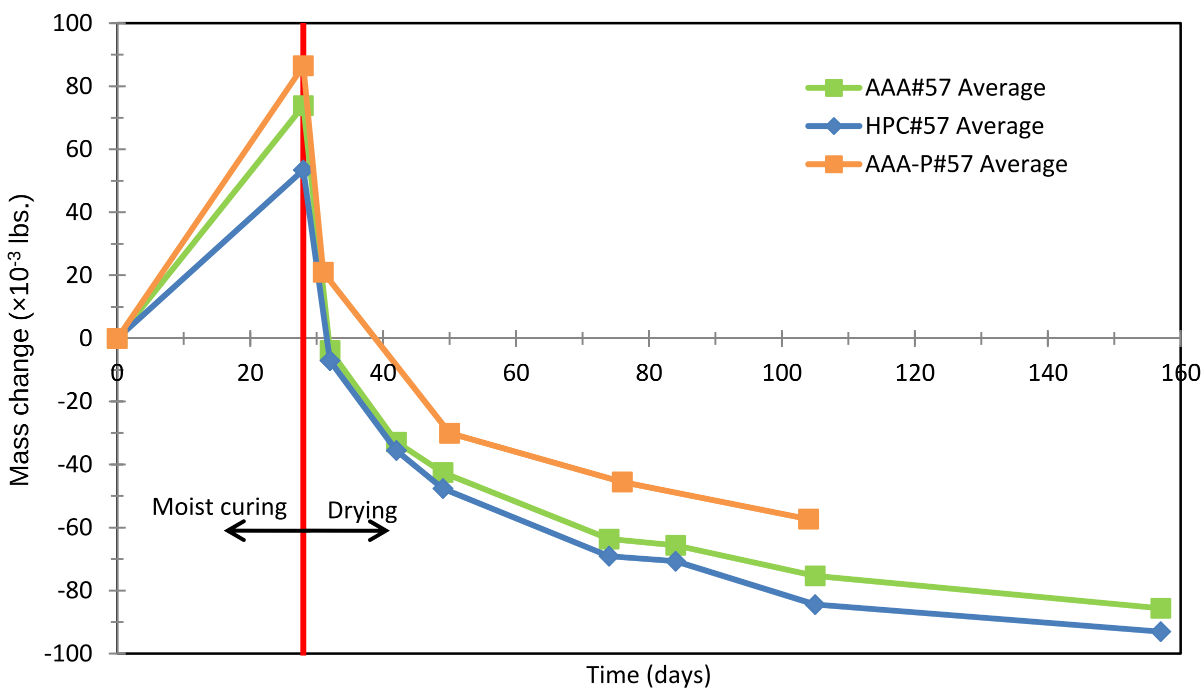


Figure 3-24: Drying mass change over time for AAA#57, HPC#57, and AAA-P#57

3.3.3.3 Restrained ring shrinkage test

A summary of the results obtained in the restrained shrinkage ring test (ASTM C 1581-09) is provided in Figures 3-25 and 3-26. The initial increase in strain observed during the first 24 hours is associated with the early-age autogenous deformation and heat of hydration under restrained condition (Radlińska et al. 2008, ASTM C 1581-09). Crack initiation (as noted in Figure 3-25) is typically seen when there is a sudden change in the strain measured from the inner surface of the steel ring, as the stresses are being released (Hossain and Weiss 2004).

Figure 3-25 shows results of the restrained ring shrinkage test (ASTM C 1581-09) for three duplicate rings from the mixture AAA#57. The average reading of four gages mounted on the inner surface of the steel ring is presented. It can be noticed that while the strain development between the rings is consistent, ring 2 cracked at the age of 11 days (at a steel strain level of approximately -35 $\mu\varepsilon$), ring 3 cracked at 15 days (at a steel strain level of approximately -34 $\mu\varepsilon$), while ring 1 remained uncracked during the duration of the study at a steel strain of approximately -42 $\mu\varepsilon$. The phenomenon of only one ring out of a few not exhibiting cracking has been observed before and does not indicate inaccuracy of the test method (Radlińska et al. 2008), but rather corresponds to inherent material variability of a heterogeneous composite material such as concrete. Appendix A presents individual rings results (data from each gage) from the restrained shrinkage (ring) test.

Figure 3-26 shows results of the restrained ring shrinkage test for three duplicate rings from the mixture HPC#57. The average reading of four gages mounted on the inner surface of the steel ring is presented. It can be noticed that the strain development between the rings is quite consistent. At 28 days, ring 1 and ring 2 have a maximum strain level of approximately -56 $\mu\varepsilon$; while ring 3 has a maximum strain level of approx. -53 $\mu\varepsilon$. The rings have not cracked as of 40 days. Appendix A presents individual rings results (data from each gage) from the restrained shrinkage (ring) test. The restrained ring test was not performed on the AAA-P#57 mixture.

According to ASTM C 1581-09, the average stress rate at cracking can be determined according to Equation 3-10. If no cracks are visible, the last day of testing can be used as the elapsed time during calculations.

$$q = \frac{G\left|\alpha_{avg}\right|}{2\sqrt{t_r}} \quad (3\text{-}10)$$

where: q = stress rate in each test specimen [psi/day]; G = a constant, 10.47 $\times 10^6$ [psi]; $\left|\alpha_{avg}\right|$ = absolute value of the average strain rate factor for each test specimen [(in./in.)/day1/2]; t_r= elapsed time at cracking or elapsed time when the test is terminated for each test specimen [days].

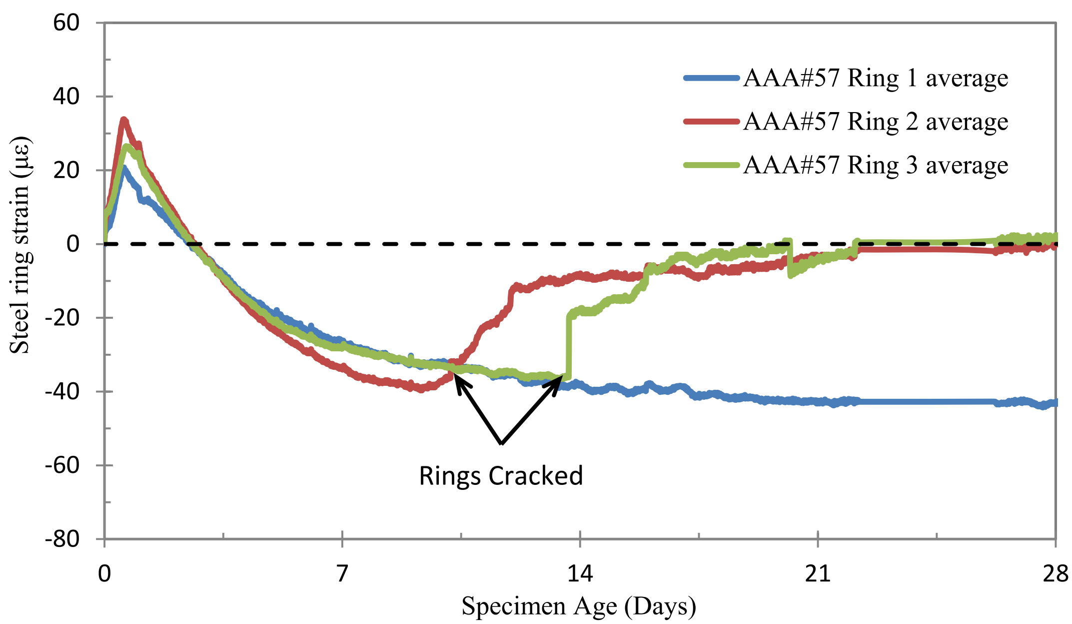


Figure 3-25: Average strain recorded for rings 1-3, mixture AAA#57

The strain rate factor, α, is the slope of the line equating the net strain (ϵ_{net}) versus the square root of time (Equation 3-11):

$$\epsilon_{net} = \alpha\sqrt{t} + k \quad (3\text{-}11)$$

where: ϵ_{net}= net strain [in./in.]; α = strain rate factor for each strain gage on the test specimen [(in./in.)/day1/2]; t = elapsed time, days; k = regression constant.

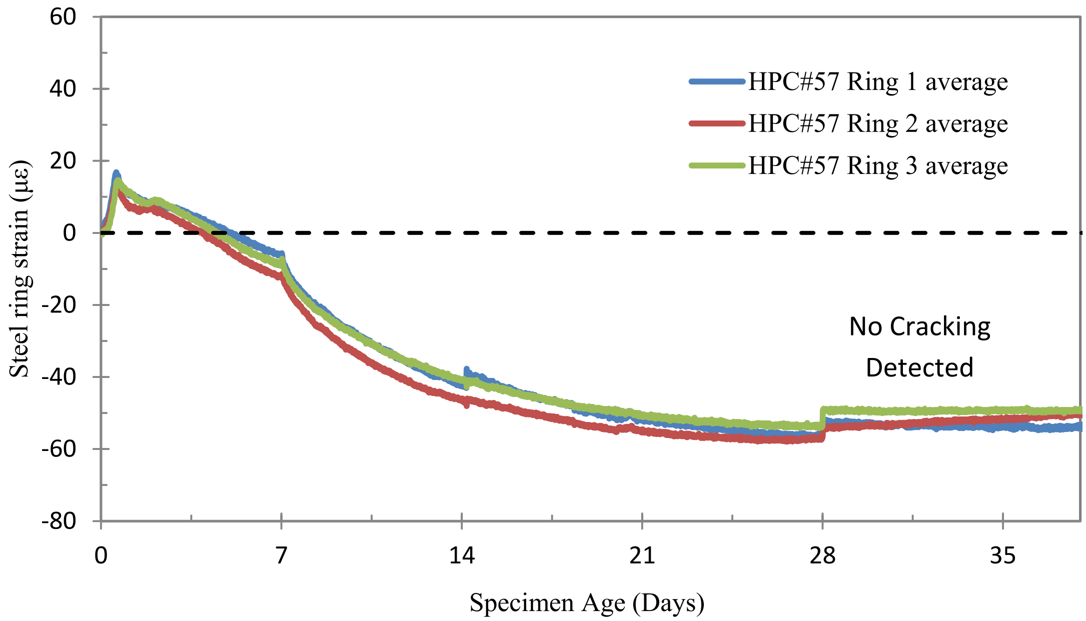


Figure 3-26: Average strain recorded for rings 1-3, mixture HPC#57

The calculated average magnitude of restrained shrinkage stress rate calculated can then be compared against ASTM C 1581-09 for cracking potential classification (Table 3-10). The average calculated stress rate for AAA#57 specimens was 28 psi/day, indicating a moderate to high potential for cracking, where cracking would be expected for specimens between the ages of 7 and 14 days old. The average calculated stress rate for HPC#57 specimens was 12 psi/day, indicating a low potential for cracking, where cracking would be expected for specimens older than 28 days.

Table 3-10: Potential for cracking classification

Net Time-to Cracking, t, (days)	Average Stress Rate, S (psi/day)	Potential for Cracking
$0 < t \leq 7$	$S \geq 50$	High
$7 < t \leq 14$	$25 \leq S < 25$	Moderate-High
$14 < t \leq 28$	$15 \leq S < 25$	Moderate-Low
$t > 28$	$S < 15$	Low

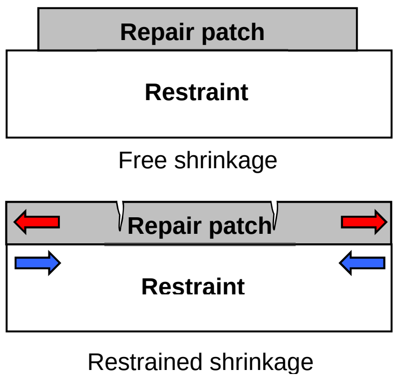
3.3.4 Deterministic Calculation of the Risk of Cracking

As outlined by Radlinska (2008), there are a variety of methods of different levels of complexity available to calculate the risk of cracking of restrained concrete elements. Below, a simple deterministic model is described that is based on calculating the elastic stress induced by thermal and shrinkage strains using Hook's law, partial relaxation of elastic stresses to account for creep, and comparing the results with the (indirect) tensile strength of concrete. As such, the risk of cracking of a concrete element can be calculated using Equation 3-12:

$$\text{Risk of cracking} = \frac{\text{Magnitude of restrained shrinkage stresses}}{\text{Tensile strength of concrete}} \qquad (3\text{-}12)$$

This is schematically illustrated in Figure 3-27, which shows stress and strength development in a restrained concrete section over time. To better match field exposure of concrete at bridge deck dams, in this work, the risk of cracking of AAA#57, HPC#57, and AAA-P#57 mixtures is calculated after 7 days of moist curing and 7 days of drying. The risk of cracking calculations are provided in Table 3-11 by accounting for thermal and drying shrinkage stresses. Since concrete was moist cured for 7 days and had w/cm higher than 0.42, autogenous shrinkage was not included in the calculations.

As illustrated in Figures 3-19, 3-20, and 3-21, the thermal strain resulting from cooling of concrete occurs primarily between 24 and 120 hours (1 to 5 days). Considering each mixture's coefficient of thermal expansion, the resulting thermal strain is 116 $\mu\varepsilon$, 113 $\mu\varepsilon$, and 112 $\mu\varepsilon$ for AAA#57, HPC#57, and AAA-P#57 mixtures, respectively. The elastic thermal stress is calculated using Hook's law by considering the 3-day elastic modulus (average age during cooling period 1 to 5 days). Since the 3-day modulus has not been measured, its value is estimated using the 28-day modulus based on correlations provided by Radlinska (2008).



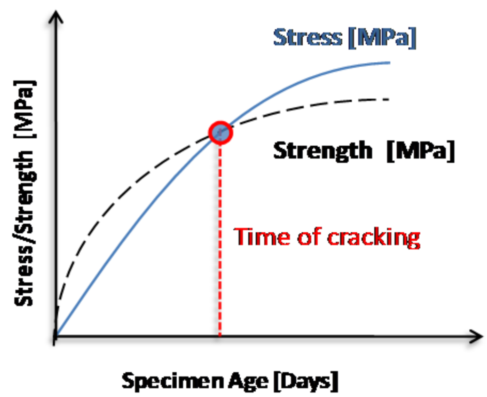


Figure 3-27: (a) Schematic illustration of shrinkage-induced cracking in concrete bridge decks, (b) time-dependent stress and strength development in concrete leading to early-age cracking

Table 3-11: Calculation of risk of cracking of concrete mixtures after 7 days of moist curing and 7 days of drying

	AAA#57	HPC#57	AAA-P#57
Thermal strain (με)	116	113	84
3day elastic modulus (lbs./in^2)	3.70×10^6	3.70×10^6	3.40×10^6
Elastic thermal stress (lbs./in^2)	429.6	418.5	285.2
7day drying shrinkage (με)	141	128	172
7day elastic modulus (lbs./in^2)	4.61×10^6	4.67×10^6	4.09×10^6
Elastic drying stress (lbs./in^2)	644.2	584.8	703.5
Total elastic stress (lbs./in^2)	1073.9	1003.4	988.68
60% of total elastic stress (lbs./in^2)	644.3	602.0	593.2
14 day estimated tensile strength (lbs./in^2)	449.4	447.7	486.7
Risk of cracking	1.434	1.345	1.219

The drying shrinkage that occurs within 7 days of drying can be estimated based on Figure 3-23 as 141 με, 128 με, and 160 με for the AAA#57, HPC#57, and AAA-P#57 mixtures. The majority of this shrinkage occurs soon after initiation of drying. To calculate the elastic drying stress, Hook's law is used along with 7-day elastic modulus of concrete. The total elastic stress is the sum of elastic thermal and drying stresses. The elastic stress values reported in Table 3-11 do not account for significant stress relaxation that can occur due to creep of concrete. An accurate calculation of stress relaxation requires complicated modeling; however, Weiss (1999) suggested that a simple first attempt could be to reduce the elastic stresses by 40%.

These stress values can now be compared against concrete's tensile strength to calculate the risk of cracking. In Table 3-11, the 14-day tensile strengths of the three mixtures are estimated based on compressive strength values using Eq. (3-9). Finally, the risk of cracking is calculated using Eq. (3-12). All three mixtures have a risk of cracking exceeding 1.000, which indicates that in the absence of reinforcing steel they will certainly crack. The risk of cracking is lower for the HPC#57 mixture, in agreement with the restrained ring test results. This is primarily due to lower thermal and shrinkage stresses in this mixture. Of the three mixtures, AAA-P#57 has the lowest risk of cracking value.

3.3.5 Rapid Chloride Permeability Test (RCPT)

The RCPT results are provided in Table 3-12. All three mixtures perform satisfactorily with low to moderate ion penetrability. PennDOT Publication 408/2007 currently does not specify a maximum allowable charge passed. The values attained are reasonable for concretes with w/cm equal to 0.43 (AAA), 0.44 (HPC), and 0.45 (AAA-P) (Wright 2012). Although AAA-P has the greatest w/cm, its incorporation of a greater percentage of GGBFS may help reduce its chloride penetration susceptibility (Whiting 1988, Stanish et al. 1997).

Table 3-12: RCPT test results

Mixture	AAA#57	HPC#57	AAA-P#57
Charge passed (Col)	1850	2130	1990
Chloride penetrability	Low	Moderate	Low

3.4.0 APPROACHES TO IMPROVE CRACKING PERFORMANCE OF CONCRETE MIXTURES

As discussed in chapter 1, early-age cracking of concrete members can be caused by (a) less than optimum concrete mixture proportions resulting in high thermal and hygral (e.g., drying, autogenous) shrinkage and low or excessively high concrete strength and stiffness; (b) less than optimum construction practices, especially improper or insufficient curing and the absence of effective ambient condition monitoring and evaporation prevention techniques to eliminate or minimize the risk of plastic shrinkage cracking; and (c) inadequate structural design considerations leading to prescribing insufficient reinforcing steel to carry thermal, hygral, and mechanical service loads. This chapter has focused on evaluating concrete mixture proportions and material properties that could contribute to a higher risk of early-age cracking. As summarized in the next section, both concrete mixtures show an overall acceptable performance without excessive heat of hydration, autogenous and drying shrinkage, or inadequate mechanical performance (e.g., excessively low or high compressive strength and stiffness). Nevertheless, there are ways to further improve the cracking performance of these mixtures, as discussed below.

Primarily, the cement paste content must be minimized (i.e., aggregate content must be maximized) to lower the heat of hydration, to reduce the coefficient of thermal expansion, and to reduce autogenous and drying shrinkage. This can be achieved by aggregate grading optimization using the Shiltstone method (1990), which is aimed at maximizing the aggregate packing. Penn State investigated the void percentage of typical aggregate gradations used by PennDOT as well as gradation suggested by literature to increase aggregate particle packing (aggregate gradations with a lower void content would therefore need less cement paste to fill the voids). This objective was completed by blending aggregates together in a 6-inch by 12-inch steel cylinder and finding the dry rodded unit weight and the remaining void content between the aggregates. The results presented in Table 3-13, show that the void content may be reduced from 31.4% for a #8 coarse aggregate to 30.8% for a #8/#89 coarse aggregate blend. These results show that the aggregate gradations used by PennDOT are adequate for reducing the cement paste content of the concrete system. Also, this information shows that PennDOT mixtures with a cementitious paste content of 33% (including air) may be reasonable. However, the field implemented concrete mixtures where the cementitious paste content is greater than 35% (including air) have too high value of cementitious paste content and need to be modified.

Table 3-13: Aggregate blend void percentages

Gradation Blend	Percentage of Voids (%)
60% #57 40% Sand	31.3
60% #8 40% Sand	31.4
30% #8 30% #89 40% Sand	30.8

In addition, given that the compressive strength of actual batched concretes were 22% to 36% higher than the minimum 28-day strength requirements dictated by structural design, the dosage of slag (GGBFS) can be increased (i.e., higher amounts of Portland cement can be replaced) to slightly reduce the early-age strength and elastic modulus with the added benefit of reducing the heat of hydration of concrete and improving the long-term strength and durability. The impact of increasing slag content on autogenous and drying shrinkage of concrete must be experimentally evaluated. Also, the use of high slag concrete in colder months may result in reduced concrete temperature and low rate of strength development. Strategies to maintain concrete temperature (e.g., use of blankets, heaters, warm mix water or aggregates) could be employed to counteract the effect of reduced heat of hydration caused by the use of slag.

Finally, shrinkage reducing admixtures (SRA) could be used which are known to be able to reduce the drying shrinkage of concrete by up to 50% (Shah et al. 1992, Folliard and Berke 1997) while maintaining other fresh and hardened properties of concrete. Alternatively, fibers (e.g., steel, polypropylene, etc.) can be used to increase the tensile strength of concrete. The main challenge in using either SRA or fibers is the associated cost of these materials that increases the total cost of concrete. In case of fibers, proper fiber type, dosage, and mixing practices must be used to ensure adequate workability of concrete and proper dispersion of fibers.

3.5.0 SUMMARY AND CONCLUSIONS

The objective of this chapter was to experimentally evaluate the performance of three concrete mixtures that are commonly used in bridge deck dam rehabilitation projects by PennDOT. These are AAA#57, HPC#57, and AAA-P#57 concretes, all utilizing AASHTO #57 coarse aggregates. The major findings based on the experiments performed on these mixtures are listed below:

- The compressive strength of AAA#57, HPC#57, and AAA-P#57 mixtures at 28 days all exceeded the required minimum strength values from structural design. The 28-day strength for AAA#57 mixtures was 6099 psi (mixture made at PSU) and 5510 to 5721 psi (mixtures used/placed at past PennDOT projects). The 28-day strength of HPC#57 was 5550 psi (mixture made at PSU). The 28-day strength of AAA-P#57 was 5120 psi (mixture made at PSU). The minimum required 28-day compressive strength was 4500 psi.
- Other mechanical properties of the three mixtures (i.e., indirect tensile strength, flexural strength, elastic modulus, Poisson's ratio) were within the ranges reported in the literature and considered acceptable.
- AAA#57 and HPC#57 showed consistent heat signatures corresponding to a heat of hydration temperature rise from 73°F to 97°F. AAA-P#57 concrete's heat evolution peaked at 95°F. The presence of slag (GGBFS) contributed to controlling the temperature rise, seen even more noticeable on the AAA-P#57 mixture considering its greater GGBFS percentage.
- AAA#57, HPC#57, and AAA-P#57 showed a low autogenous shrinkage and moderate values of drying shrinkage, in agreement with literature results for mixtures with similar w/cm and containing GGBFS. AAA-P#57 shows a larger drying shrinkage than the other two mixtures.
- The restrained ring test showed that HPC#57 has a superior resistance against restrained shrinkage cracking. HPC#57 rings did not crack up to 40 days of testing and showed low potential for cracking based on ASTM C1581 calculation procedure. In contrast, two of the AAA#57 rings cracked at 11 and 15 days after casting.

- Deterministic calculation of the risk of cracking also shows a higher cracking risk for the AAA mixture due to its higher drying shrinkage and higher coefficient of thermal expansion. The AAA-P mixture has the lowest risk of cracking using the same formula.
- The rapid chloride permeability test showed slightly better performance for the AAA mixture, possibly due to its lower w/cm.

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CHAPTER 4 (TASK 4)

Summary, Conclusions, and Recommendations

4.1.0 SUMMARY

The main objective of this research project was to identify the causes of longitudinal cracking in newly placed concrete deck segments adjacent to bridge deck expansion dam replacements. The project was performed in five tasks as follows:

- **Task 1** provided a comprehensive review of the literature related to the causes of early-age cracking in concrete bridge decks which, also includes cracking associated with bridge dam replacements and other types of concrete repair sections. In addition, a national survey of state transportation agencies was performed regarding their practices to mitigate early-age cracking of concrete.
- **Task 2** presented and discussed the results of a review of the relevant PennDOT construction specifications and provided a comparison with literature recommendations to determine the adequacy of the existing specifications to minimize the risk of cracking. Also, in this task, a review of the design and construction documentation associated with three past and two active projects was performed. This review included the concrete mix design, the steel reinforcing bar design, and the construction practices implemented by contractors. The results were compared against PennDOT specifications and literature recommendations.
- **Task 3** was an experimental evaluation of the quality and cracking risk of three common concrete mixtures used by PennDOT in bridge deck rehabilitation projects. These include concrete mixtures AAA#57, HPC#57, and AAA-P#57. The experimental assessment included measurement of the fresh properties, mechanical properties, shrinkage and temperature development, and the durability properties of each mixture as well as calculation of the risk of cracking of these mixtures. Methods for improving the cracking resistance of mixtures are also discussed.

- **Tasks 4 and 5** included preparation of a final project report and an oral presentation of the findings.

The conclusions based on Tasks 1 through 3 are offered at the end of the corresponding chapters (sections 1.3.0, 2.5.0, and 3.5.0). This chapter provides the most important and overriding conclusions, as well as recommendations to PennDOT, considering modification of the relevant sections of the *Specifications Publications 408* as well as proper supervision and QA practices during construction operations.

4.2.0 MAIN CONCLUSIONS

Based on the literature review, the research team identified three potential causes of cracking in concrete bridge decks to be investigated during this study. The three potential causes to be investigated were identified as follows:

(1) Less than optimum concrete mixture proportions resulting in high shrinkage, high thermal contraction, and low or excessively high concrete strength and stiffness;

(2) Inadequate structural design considerations, leading to prescribing insufficient reinforcing steel to carry temperature, shrinkage, and mechanical service loads;

(3) Less than optimum construction practices, especially improper or insufficient curing and absence of effective ambient condition monitoring and water evaporation prevention techniques to eliminate or minimize the risk of plastic shrinkage cracking.

Based on the results of the investigation of the three potential causes for cracking mentioned above, it is concluded that:

- The existing PennDOT bridge deck concrete mixtures AAA#57, AAA-P#57, and HPC#57 show acceptable performance if they are properly placed, consolidated, finished, and cured.
- The review of the three past projects in District 3-0 show the design of steel reinforcement in these projects is adequate with respect to shrinkage and temperature steel and should not result in early-age cracking.

- The review of the past and active projects reveal significant deficiencies with respect to proper water curing (both method and duration) as well as failure to implement methods to actively monitor and eliminate the risk of plastic shrinkage cracking. This is the most likely cause of early cracking observed in the newly rehabilitated bridge decks.

4.3.0 RECOMMENDATIONS TO PENNDOT

4.3.1 Suggested Modifications to PennDOT Specifications Publication 408

The suggested modifications to PennDOT specifications are marked with RED. Other important language is underlined.

SECTION 704: CEMENT CONCRETE

704.1 (b) should read as follows:

(b) Material.

- Cement—Section 701
- Fine Aggregate, Type A—Section 703.1
- Coarse Aggregate, Type A, No. 57, No. 67 or No. 8 (Stone, Gravel, or Slag)—Section 703.2[(1)]
- Water—Section 720.1
- Admixtures—Section 711.3
- Pozzolan—Section 724

Note (1): Blending of two or more coarse aggregate sizes are permitted if it results in improved aggregate packing and reduced binder content of concrete without sacrificing the required fresh and hardened properties of concrete.

TABLE A: requirements for AAAP mixtures should read as follows (according to the most current version of Publication 408, only AAAP mixture is allowed for bridge deck construction):

Class of Concrete	Use	Cement Factor (lbs./yd^3)		Max allowable w/cm	Min Design Compressive Strength (psi)			Proportions Coarse Aggregate Solid Volume (ft^3/yd^3)	28-Day Structural Design Comp. Strength (psi)
					Days				
		Min	Max		3	7	28		
AAA-P	Bridge Decks	560	~~752~~ 634.5	0.45	-	3000	4000	-	4000

The rest of **704.1 (b)** remains unchanged.

704.1 (c) bullet 2. should read as follows:

2. Cement Factor. For all classes of concrete, use the minimum cement factor (cement or cement and pozzolan combined) specified in Table A, except as follows:

Portland cement may be replaced with pozzolan (flyash or ground granulated blast furnace slag) weighing as much as or more than the Portland cement replaced. If pozzolan is used, do not place flyash and ground granulated blast furnace slag in the same mix. The maximum limit of the cement factor ~~may~~ must not be waived ~~if~~ when pozzolan is added to the mix ~~provided the Portland cement portion does not exceed the maximum cement factor specified~~. If flyash is used, the Portland cement portion may be reduced by a maximum of 15%. If ground granulated blast furnace slag is used, the Portland cement portion may be reduced by a minimum of 25% to a maximum of 50%. If Mechanically Modified Pozzolan-Cement combinations are used, the Portland cement portion may be reduced by a maximum of 50%.

The rest of **704.1 (c) bullet 2** remains unchanged.

704.1 (c) bullet 4. should read as follows:

4. Mix Design Acceptance. Submit a copy of each completed mix design to the Representative before its use in the work. The Department reserves the right to review any design through plant production before its use in Department work at no additional cost to the Department. The concrete design submitted for review is required to comply with the specified concrete class requirements, supported by slump, air content, and compressive strength test data according to Bulletin 5.

The Department will accept concrete designs on the basis of the 7-day strength tests (Class high early strength (HES) may be accepted on the basis of 3-day strength tests); however, conduct 28-day tests to show the potential of the design mix. The Department may also accept designs based on the 28-day tests. For bridge deck applications, the 28-day compressive strength test results must not exceed the 28-day structural design compressive strength by more than 1000 psi.

The rest of **704.1 (c) bullet 4** remains unchanged.

704.1 (d) bullet 4.a should read as follows:

4.a QC Sampling and Testing of Plastic Concrete. Select an appropriate slump value that will provide a workable mix for the construction element. The Contractor's technician must have a copy of the Department reviewed QC Plan in their possession during testing and must be aware of the target slump for the structural element being placed. Do not exceed the following slump upper limits:

Type of Mix	**Slump Upper Limit**
without water reducing admixtures	5 inches
with water reducing admixtures	6 1/2 inches
with high range water reducing admixtures (superplasticizers)	8 inches
mixes specified in Section 704.1(h) (except tremie concrete as specified in Section 1001.2(j))	2 1/2 inches
bridge deck concrete	4 inches

The rest of **704.1 (d) bullet 4.a** remains unchanged.

704.1 (d) bullet 4.b.2 should read as follows:

4.b.2 28-Day QC Compressive Strength. If the 28-day QC compressive strength test result is greater than or equal to the 28-day minimum mix design compressive strength specified in Table A, acceptance of the concrete lot will be based on the compressive strength testing of acceptance cylinders as specified in Section 704.1(d)5. For AAAP concrete mix used for bridge decks, the 28-day QC compressive strength test result must not exceed the 28-day structural design compressive strength by more than 1000 psi.

The rest of **704.1 (d) bullet 4.b.2** remains unchanged.

SECTION 1001: CEMENT CONCRETE STRUCTURES

1001.3(k) bullet 6 should read as follows:

6. Bridge Decks. At least 2 weeks before concrete deck placement, schedule a deck preplacement meeting to review the specification, method and sequence of placing deck concrete, quality control testing, and method of protective measures, to control the concrete evaporation rate. Place concrete at a concrete temperature of between 50F and 80F. Provide the necessary equipment and determine the evaporation rate before starting deck placement and every hour during the placement. Do not exceed an evaporation rate of ~~0.15~~ 0.10 pounds per square foot per hour. The allowable Evaporation Rate for exposed finished concrete is determined by ACI 305R-91, Figure 2.1.5 Have readily available at the bridge deck placement site, all remediation equipment and procedures as submitted and approved at the deck preplacement meeting before starting the placement. If the value is exceeded, stop concrete placement until protective measures are taken to reduce the values to an acceptable level. Fog cure misting is an acceptable method to mitigate an excessive evaporation rate. Use high pressure equipment that generates at least1,204 pounds per square inch at 2.19 gallons per minute, or with low pressure equipment having nozzles capable of supplying a maximum flow rate of 1.66 gallons per minute. Use nozzles that atomize droplets and can keep a large surface damp without causing water deposits. Apply the fog over the entire placement that is not covered by wet burlap. Do not leave concrete exposed for extended duration. Place concrete 5 feet to 8 feet ahead of finishing machine to prevent any premature concrete drying.

Fresh concrete surface must never be exposed to drying for more than 15 minutes. If the concrete placement and finishing operations are temporarily stopped before the final finishing, concrete must be protected using fog spray or other adequate technique to prevent drying of the surface of the newly placed concrete.

For rigid frame decks, place the concrete from the center of the span toward each leg or abutment simultaneously. Continuously check falsework or supporting beams so the concrete, as placed, meets the lines and grades indicated. Keep wedges and blocking tight during placement of the concrete.

Use a placing sequence for continuous spans, as indicated.

Unless allowed in writing by the District Executive, do not allow truck mixers, truck agitators, or other heavy motorized equipment on the deck spans in which concrete is being placed.

If it is necessary to stop operations, due to weather or operational conditions, provide bulkheads at the work site, and place them as directed. Remove bulkheads before resuming concrete placement operations.

Obtain acceptance of changes or additions to indicated construction joints, before incorporating into the work.

Use motorized, mechanical finishing equipment. Submit a sketch to the Inspector-in-Charge, describing the equipment and showing complete details of supports for the equipment.

Adjust the deck openings at expansion joints and at expansion dams at the time concrete is placed to provide the openings indicated at 68F under full dead load.

Vibrating screeds may be used, with the written permission of the District Executive. Vibrating screeds are to be power-vibrated and moved by means of a positive, power-operated apparatus, but are not to be a substitute for high-frequency vibrators. Hand-finishing methods will be allowed outside mechanically screeded areas and to a placed bulkhead in cases of power equipment failures.

Use strike-off finishing machines or screeds large enough to finish the full width of deck between curbs or between longitudinal construction joints, or between both.

When strike-off finishing machines are used, support the wheels above the pavement surface on temporary rails, supported on non-deflecting forms or other horizontal structural devices. Support vibrating screeds on temporary pipe guides or on-grade angles. Use adjustable finishing machine supports or vertical supports for screed guides. Fix supports during finishing, at intervals to limit deflection to not more than 1/8 inch in 10 feet. Use supports that are removable to at least 2 inches below the surface with a minimum disturbance of concrete. Fill voids left upon removal of screed guides and supports with nonstaining, nonshrinking mortar, after the deck concrete has reached its initial set.

Do not allow screed or runway supports to bear on the forms, unless direct undersupport is provided to prevent form damage or deflection. Do not discharge concrete near side laps or at midspan of the corrugated sheets, to a depth greater than 10 inches above the top of the forms. Do not discharge concrete in a manner that causes excessive concentrated construction loads.

Place concrete, at a minimum rate of 20 linear feet of deck per hour, in a longitudinal direction, except for reinforced concrete slabs and rigid frames.

Vibrate the concrete to prevent honeycombing and voids, especially at construction joints, expansion joints, valleys, and ends of form sheets. Obtain acceptance of placing sequences, procedures, and mixes before placing concrete.

Repair or replace damaged material.

Conduct final finishing operations immediately behind the finishing machines or screeds from work bridges of rigid construction, not in contact with the surface of the concrete, set on rails, and easily moved. Finish with a 10-foot, long-handled straightedge to achieve a smooth surface. Make one pass of the float if after the finishing machine operations the concrete surface remains open. Do not overfinish. Fog misting equipment is allowed on the finishing machine to maintain the evaporation rate below the allowable value.

Perform straightedge testing and surface correction as specified in Section 501.3(k)3 while the concrete is workable. After completing the straightedge testing and surface corrections, before the concrete becomes nonplastic, texture the surface as specified in Section 501.3(k)4. Immediately after texturing operations are completed, perform intermediate curing as per Section 1001.3(p) 3.c. Cure the deck as specified in Section 1001.3(p)3.b for a minimum of 14 days. Maintain wet burlap application within 10 feet to 18 feet behind the finishing equipment at all times. Minimal marking of the concrete is allowed. Following cure, test the surface again, as specified in Section 501.3(o).

For bridge decks placed between September 1 and March 1, apply a penetrating sealer as specified in Section 1019.3(c) 2.

If directed to facilitate inspection, remove at least one section of permanent forms, at a location directed, for each span of every bridge in the project. After the deck concrete has been in place for a minimum period of 2 days, test the concrete by sounding with a hammer, where directed. If hollow sounding areas are found, and if directed, remove the forms for the Representative's inspection after the concrete has attained adequate strength. The forms need not be replaced. Repair the adjacent metal forms and supports in order to present a neat appearance. Remove or repair unsatisfactory concrete. Provide facilities for the safe and convenient conduct of the inspection.

1001.3(v) should read as follows:

(v) Bridge Approach Slabs. Construct as shown on the Contract Drawings and in accordance with Section 505.3. Water cure as specified in Section 1001.3(p)3.b for a minimum of 7 days.

4.3.2 Other Recommendations

- PennDOT must strictly enforce its specifications regarding active monitoring of the construction site ambient conditions (temperature, relative humidity, wind speed). PennDOT should require contractors to include in their QC plan, the methods that the contractor will use to regularly monitor the site ambient conditions during placement of concrete, and the type of remediation techniques that the contractor will utilize immediately if the estimated evaporation rate of water exceeds the specification limits (0.1 lbs/ft^{2}hr). PennDOT QA personnel must monitor the contractor's compliance with these rules during construction.
- PennDOT QA personnel must ensure the contractor's compliance with proper method of water curing of concrete bridge deck. The burlaps must never dry out during the water curing process.

Appendix A

AASHTO LRFD Bridge Design Specifications for Transverse Strength

(Barker and Puckett 2007; AASHTO LRFD 2012; ACI 318-11)

Temperature and shrinkage steel requirements

Temperature and shrinkage steel area requirements are provided by both ACI and AASHTO. These methods are described below.

ACI 318-11 requires temperature and shrinkage steel requirements to be calculated using Equation A1:

$$A_s(t) = 0.0018bh \tag{A1}$$

where: $A_s(t)$ = Area of steel; b = deck length; and h = deck depth.

AASHTO *LRFD Bridge Design Specifications* require temperature and shrinkage steel to be calculated using Equation A2:

$$A_s(t) = \frac{1.30bh}{2(b+h)f_y} \tag{A2}$$

as well as satisfy the inequality shown as Equation A3:

$$0.11 > A_s(t) < 0.60 \tag{A3}$$

Reinforcing bar spacing shall not exceed three (3) times the component thickness (or 18 inches), 12 inches for walls and footings greater than 18 inches, and 12 inches for other components greater than 36 inches thick.

Both of these calculations are shown in Tables A1 through A3.

Determining negative moment reinforcement

Maximum negative live load moment per unit width (kip-ft./ft.) is reported directly from Table A4-1 in Appendix A4 of the AASHTO *LRFD Bridge Design Specifications.* Maximum negative dead load moment per unit width, kip-ft/ft, was determined using finite element analysis (SAP2000). Equation A4 is therefore used to determine the negative moment for reinforcement design:

$$M_{-U} = 1.75 \times 1.33 \times 1.25 \times M_{-LL} + 1.5 \times M_{-DL} \qquad \text{(A4)}$$

where: M_{-U} = ultimate factored moment; M_{-LL} = negative live load moment per unit width (kip-ft/ft); and M_{-DL} = negative dead load moment per unit width (kip-ft/ft). 1.25 is applied to the M_{-LL} to determine the HS-25 truck factored live load

Depth to negative reinforcing bars (d) is determined by subtracting the top concrete cover and half of the nominal rebar diameter from the deck depth. Therefore the area of steel required (A_{-s}) for negative reinforcing can be determined from the following quadratic equation (Equation A5)

$$M_{-n} = 0.9 \times A_{-s} \times f_y \times d \times (1 - 0.6 \times (\frac{A_{-S}}{bd}) \times (\frac{f_y}{f'_c})) \qquad \text{(A5)}$$

where: M_{-n} = nominal load carrying capacity considering negative moment; and A_{-s}= steel reinforcing bar area.

Values are reported in Tables A4-A25.

Determining positive moment reinforcement

Maximum positive live load moment per unit width (kip-ft./ft.) is reported directly from Table A4-1 in Appendix A4 of the AASHTO *LRFD Bridge Design Specifications.* Maximum positive dead load moment per unit width, kip-ft/ft, was determined using finite element analysis (SAP2000). Equation A6 is therefore used to determine the negative moment for reinforcement design:

$$M_{+U} = 1.75 \times 1.33 \times 1.25 \times M_{+LL} + 1.5 \times M_{+DL} \quad \text{(A6)}$$

where: M_{+U} = ultimate factored moment; M_{+LL} = negative live load moment per unit width (kip-ft/ft); and M_{+DL} = negative dead load moment per unit width (kip-ft/ft). 1.25 is applied to the M_{+LL} to determine the HS-25 truck factored live load

Depth to positive reinforcing bars (*d*) is determined by subtracting the top concrete cover and half of the nominal rebar diameter from the deck depth. Therefore the area of steel required (A_{+s}) for positive reinforcing can be determined from the following quadratic equation (Equation A5)

$$M_{+n} = 0.9 \times A_{+s} \times f_y \times d \times (1 - 0.6 \times (\frac{A_{+s}}{bd}) \times (\frac{f_y}{f'_c})) \quad \text{(A5)}$$

where: M_{+n} = nominal load carrying capacity considering positive moment; and A_{+s}= steel reinforcing bar area.

Values are reported in Tables A4-A25.

Control of Cracking by Reinforcing bars

PennDOT has used a reinforcing bar distance of 6 inches; therefore, the following calculations shown in Tables A4 through A25 back-calculate the spacing that would be necessary given the reinforcement ratio provided by #5 bars at **6 inch spacing.** The following steps are outlined in AASHTO LRFD Bridge Design Specifications to calculate the control of cracking. The equivalent moment strips (positive and negative) are determined by Equations A8 and A9, respectively.

$$+M = 26.0 + 6.6S \tag{A8}$$

$$-M = 48.0 + 3.0S \tag{A9}$$

where: M = Equivalent interior strips for main reinforcement perpendicular to traffic (positive and negative); S = center to center spacing of girders

From here, the ultimate service moment can be calculated using Equation A10

$$M_S = 1.00 \times M_{DL} + 1.00 \times 1.33 \times M_{LL} \tag{A10}$$

where: M_s= Ultimate service factored moment; M_{LL} = Negative live load moment per unit width (kip-ft/ft); and M_{DL}= Negative dead load moment per unit width (kip-ft/ft)

Next, the multiplier (k) is used to establish the depth of concrete in compression in the elastic range using Equation A11. Please review Figure A1 for a schematic.

$$k = \sqrt{2\rho n + (\rho n)^2} - \rho n \tag{A11}$$

where: k = multiplier used to establish the depth of concrete in compression in the elastic range; ρ = Steel reinforcement ratio; and n = Modular ratio

The multiplier (k) is used in part to calculate the distance between the resultants of the internal compressive and tensile forces on a cross section (jd). Equation A12 shows the lever arm jd according to the straight line stress distribution.

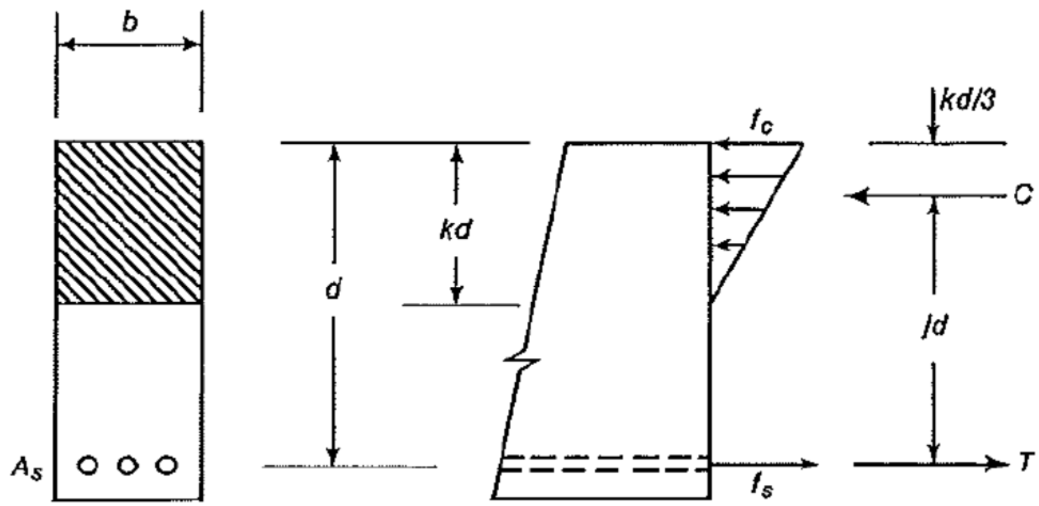

Figure A3: Stress distribution in straight line theory (Courtesy of Wight and MacGregor 2009)

$$jd = d - \frac{kd}{3} \tag{A12}$$

where: jd = distance between the resultants of the internal compressive and tensile forces on a cross section; and k = multiplier used to establish the depth of concrete in compression in the elastic range

This j value is then used to determine the stress in the reinforcement at service load f_{ss} in Equation A13

$$f_{SS} = \frac{M_S \times b}{A_s \times jd} \tag{A13}$$

where: f_{ss} stress in the reinforcement at service load; jd = distance between the resultants of the internal compressive and tensile forces on a cross section; M_s = Ultimate service factored moment; b = Steel spacing provided (6 inches); and A_s = Area of steel provided (#5 bars have an area of 0.31in^2).

The exposure factor (γ_e) is determined conservatively to be 0.75 since corrosion and cracks are not acceptable. Therefore the cracking limitation (S) can be calculated using Equation A14.

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_{ss}} - 2d_c \qquad \text{(A14)}$$

where: s = minimum reinforcement spacing for cracking control; d_c = thickness of concrete cover measured from the extreme fiber in tension to center of the flexural reinforcement located closest; f_{ss} = stress in the reinforcement at service load, γ_e = exposure factor (0.75); and β_s = factor to account for the effect of cracking and confining reinforcement on the effective compressive strength of the concrete

These calculations are shown in Tables A4 through A25.

Table A1: PennDOT project 180-058 shrinkage and temperature steel calculations

* Half of each bar amount on top and bottom and each side of the strip

Job #	S' #	Direction	Deck Depth (in)	Overlay?	Deck Length to be Demolished (ft)	Bridge Area Repaired (in^2)	Reinforcing Bar #
180-058	S-27091	Westbound Pier 1	7.50	No	5.00	450	5
							6
180-058	S-27091	Westbound Pier 2	7.50	No	5.00	450	5
							6
180-058	S-27091	Westbound Pier 3	7.50	No	5.00	450	5
							6
180-058	S-27091	Eastbound Pier 1	8.00	No	5.00	480	5
							6
180-058	S-27091	Eastbound Pier 2	8.00	No	5.00	480	5
							6
180-058	S-27091	Eastbound Pier 3	8.00	No	5.00	480	5
							6
180-058	S-27329	Pier	7.50	No	5.00	450	5
							5
							6
180-058	S-27165	Westbound Pier	7.50	No	5.00	450	5
							6
180-058	S-27165	Eastbound Pier	7.50	No	5.00	450	5
							6
180-058	S-27187	Westbound Pier 1 (Spans 1)	8.50	Yes (1.5' Latex)	2.50	255	5
							6
180-058	S-27187	Westbound Pier 1 (Span 2)	8.00	Yes (1.5' Latex)	2.50	240	5
							6
180-058	S-27187	Westbound Pier 2 (Spans 3)	8.50	Yes (1.5' Latex)	2.50	255	5
							6
180-058	S-27187	Westbound Pier 2 (Span 2)	8.00	Yes (1.5' Latex)	2.50	240	5
							6
180-058	S-27187	Eastbound Pier 1 (Spans 1)	8.00	Yes (1.5' Latex)	2.50	240	5
							6

* Half of each bar amount on top and bottom and each side of the strip

Job #	S' #	Direction	Amount of Bars	Bar Area (in^2)	bar area/foot of deck (in^2/ft)	Temp/Shrinkage ACI (0.0018bh) (in^2/ft)	LRFD temp/shrinkage (1.30bh/2(b+h)fy(in^2/ft)
180-058	S-27091	Westbound Pier 1	12	3.72	1.45	0.027	0.072
			8	3.52			
180-058	S-27091	Westbound Pier 2	12	3.72	1.45	0.027	0.072
			8	3.52			
180-058	S-27091	Westbound Pier 3	12	3.72	1.45	0.027	0.072
			8	3.52			
180-058	S-27091	Eastbound Pier 1	12	3.72	1.45	0.029	0.076
			8	3.52			
180-058	S-27091	Eastbound Pier 2	12	3.72	1.45	0.029	0.076
			8	3.52			
180-058	S-27091	Eastbound Pier 3	12	3.72	1.45	0.029	0.076
			8	3.52			
180-058	S-27329	Pier	6	1.86	0.74	0.027	0.072
			6	1.86			
			8	3.52			
180-058	S-27165	Westbound Pier	12	3.72	1.45	0.027	0.072
			8	3.52			
180-058	S-27165	Eastbound Pier	12	3.72	1.45	0.027	0.072
			8	3.52			
180-058	S-27187	Westbound Pier 1 (Spans 1)	6	1.86	1.45	0.031	0.072
			4	1.76			
180-058	S-27187	Westbound Pier 1 (Span 2)	6	1.86	1.45	0.029	0.068
			4	1.76			
180-058	S-27187	Westbound Pier 2 (Spans 3)	6	1.86	1.45	0.031	0.072
			4	1.76			
180-058	S-27187	Westbound Pier 2 (Span 2)	6	1.86	1.45	0.029	0.068
			4	1.76			
180-058	S-27187	Eastbound Pier 1 (Spans 1)	6	1.86	1.45	0.029	0.068
			4	1.76			

* Half of each bar amount on top and bottom and each side of the strip

Job #	S' #	Direction	Deck Depth (in)	Overlay?	Deck Length to be Demolished (ft)	Bridge Area Repaired (in^2)	Reinforcing Bar #
180-058	S-27187	Eastbound Pier 1 (Span 2)	7.50	Yes (1.5' Latex)	2.50	225	5
							6
180-058	S-27187	Eastbound Pier 2 (Spans 3)	8.00	Yes (1.5' Latex)	2.50	240	5
							6
180-058	S-27187	Eastbound Pier 2 (Span 2)	7.50	Yes (1.5' Latex)	2.50	225	5
							6

* Half of each bar amount on top and bottom and each side of the strip

Job #	S' #	Direction	Amount of Bars	Bar Area (in^2)	bar area/foot of deck (in^2/ft)	Temp/Shrinkage ACI (0.0018bh) (in^2/ft)	LRFD temp/shrinkage (1.30bh/2(b+h)fy(in^2/ft)
180-058	S-27187	Eastbound Pier 1 (Span 2)	6	1.86	1.45	0.027	0.065
			4	1.76			
180-058	S-27187	Eastbound Pier 2 (Spans 3)	6	1.86	1.45	0.029	0.068
			4	1.76			
180-058	S-27187	Eastbound Pier 2 (Span 2)	6	1.86	1.45	0.027	0.065
			4	1.76			

Table A2: PennDOT project 180-044 shrinkage and temperature steel calculations

* Half of each bar amount on top and bottom and each side of the strip

Job #	S' #	Direction	Deck Depth (in)	Overlay?	Deck Length to be Demolished (ft)	Bridge Area Repaired (in^2)	Reinforcing Bar #
180-044	S-26130	Westbound Pier 1	8.50	No	8.00	816	5
							6
180-044	S-26130	Westbound Pier 2	8.50	No	8.00	816	5
							6
180-044	S-26130	Eastbound Pier 1	8.50	No	8.00	816	5
							6
180-044	S-26130	Eastbound Pier 2	8.50	No	8.00	816	5
							6
180-044	S-26163	Westbound Pier 1	8.00	No	5.00	480	5
							6
180-044	S-26163	Westbound Pier 2	8.00	No	5.00	480	5
							6
180-044	S-26163	Eastbound Pier 1	7.50	No	5.00	450	5
			\				6
180-044	S-26163	Eastbound Pier 2	7.50	No	5.00	450	5
							6
180-044	S-26129	Westbound Pier 1 (Span 1)	8.00	No	2.50	240	5
							6
180-044	S-26129	Westbound Pier 1 (Span 2)	7.50	No	2.50	225	5
							6
180-044	S-26129	Westbound Pier 2 (Span 2)	7.50	No	2.50	225	5
							6
180-044	S-26129	Westbound Pier 2 (Span 3)	8.00	No	2.50	240	5
							6
180-044	S-26129	Eastbound Pier 1 (Span 1)	8.50	No	2.50	255	5
							6

* Half of each bar amount on top and bottom and each side of the strip

Job #	S' #	Direction	Amount of Bars	Bar Area (in^2)	bar area/foot of deck (in^2/ft)	Temp/Shrinkage ACI (0.0018bh) (in^2/ft)	LRFD temp/shrinkage (1.30bh/2(b+h)fy(in^2/ft)
180-044	S-26130	Westbound Pier 1	24	7.44	1.37	0.031	0.085
			8	3.52			
180-044	S-26130	Westbound Pier 2	24	7.44	1.37	0.031	0.085
			8	3.52			
180-044	S-26130	Eastbound Pier 1	24	7.44	1.37	0.031	0.085
			8	3.52			
180-044	S-26130	Eastbound Pier 2	24	7.44	1.37	0.031	0.085
			8	3.52			
180-044	S-26163	Westbound Pier 1	12	3.72	1.45	0.029	0.076
			8	3.52			
180-044	S-26163	Westbound Pier 2	12	3.72	1.45	0.029	0.076
			8	3.52			
180-044	S-26163	Eastbound Pier 1	12	3.72	1.45	0.027	0.072
			8	3.52			
180-044	S-26163	Eastbound Pier 2	12	3.72	1.45	0.027	0.072
			8	3.52			
180-044	S-26129	Westbound Pier 1 (Span 1)	8	2.48	1.70	0.029	0.068
			4	1.76			
180-044	S-26129	Westbound Pier 1 (Span 2)	8	2.48	1.70	0.027	0.065
			4	1.76			
180-044	S-26129	Westbound Pier 2 (Span 2)	6	1.86	1.45	0.027	0.065
			4	1.76			
180-044	S-26129	Westbound Pier 2 (Span 3)	6	1.86	1.45	0.029	0.068
			4	1.76			
180-044	S-26129	Eastbound Pier 1 (Span 1)	8	2.48	1.70	0.031	0.072
			4	1.76			

* Half of each bar amount on top and bottom and each side of the strip

Job #	S' #	Direction	Deck Depth (in)	Overlay?	Deck Length to be Demolished (ft)	Bridge Area Repaired (in^2)	Reinforcing Bar #
180-044	S-26129	Eastbound Pier 1 (Span 2)	7.50	No	2.50	225	5
							6
180-044	S-26129	Eastbound Pier 2 (Span 2)	7.50	No	2.50	225	5
							6
180-044	S-26129	Eastbound Pier 2 (Span 3)	8.00	No	2.50	240	5
							6

* Half of each bar amount on top and bottom and each side of the strip

Job #	S' #	Direction	Amount of Bars	Bar Area (in^2)	bar area/foot of deck (in^2/ft)	Temp/Shrinkage ACI (0.0018bh) (in^2/ft)	LRFD temp/shrinkage (1.30bh/2(b+h)fy(in^2/ft)
180-044	S-26129	Eastbound Pier 1 (Span 2)	8	2.48	1.70	0.027	0.065
			4	1.76			
180-044	S-26129	Eastbound Pier 2 (Span 2)	8	2.48	1.70	0.027	0.065
			4	1.76			
180-044	S-26129	Eastbound Pier 2 (Span 3)	8	2.48	1.70	0.029	0.068
			4	1.76			

Table A3: PennDOT project 15-7PP shrinkage and temperature steel calculations

* Half of each bar amount on top and bottom and each side of the strip

Job #	S' #	Direction	Deck Depth (in)	Overlay?	Deck Length to be Demolished (ft)	Bridge Area Repaired (in^2)	Reinforcing Bar #
15-7PP	S-27071	Eastbound Pier 1 (Span 1)	8.50	Yes (1.5' Latex)	2.25	229.5	5
							6
15-7PP	S-27071	Eastbound Pier 2 (Span 1)	8.50	Yes (1.5' Latex)	2.25	229.5	5
							6
15-7PP	S-27205	Pier 1 (Span 1)	9.25	No	2.00	222	5
							6
15-7PP	S-27205	Pier 1 (Span 2)	8.75	No	2.00	210	5
							6
15-7PP	S-27205	Pier 2 (Span 2)	8.75	No	2.00	210	5
							6
15-7PP	S-27205	Pier 2 (Span 3)	9.25	No	2.00	222	5
							6
15-7PP	S-27206	Pier 1 (Span 1)	9.25	No	2.00	222	5
							6
15-7PP	S-27206	Pier 1 (Span 2)	8.75	No	2.00	210	5
							6
15-7PP	S-27206	Pier 2 (Span 2)	8.75	No	2.00	210	5
							6
15-7PP	S-27206	Pier 2 (Span 3)	9.25	No	2.00	222	5
							6
15-7PP	S-27207	Pier 1 (Span 1)	9.75	No	2.00	234	5
							6
15-7PP	S-27207	Pier 1 (Span 2)	9.25	No	2.00	222	5
							6
15-7PP	S-27207	Pier 2 (Span 2)	9.25	No	2.00	222	5
							6
15-7PP	S-27207	Pier 2 (Span 3)	9.75	No	2.00	234	5
							6

* Half of each bar amount on top and bottom and each side of the strip

Job #	S' #	Direction	Amount of Bars	Bar Area (in^2)	bar area/foot of deck (in^2/ft)	Temp/Shrinkage ACI (0.0018bh) (in^2/ft)	LRFD temp/shrinkage (1.30bh/2(b+h)fy(in^2/ft)
15-7PP	S-27071	Eastbound Pier 1 (Span 1)	4	1.24	1.33	0.031	0.070
			4	1.76			
15-7PP	S-27071	Eastbound Pier 2 (Span 1)	4	1.24	1.33	0.031	0.070
			4	1.76			
15-7PP	S-27205	Pier 1 (Span 1)	3	0.93	1.57	0.033	0.072
			5	2.2			
15-7PP	S-27205	Pier 1 (Span 2)	3	0.93	1.57	0.032	0.069
			5	2.2			
15-7PP	S-27205	Pier 2 (Span 2)	3	0.93	1.57	0.032	0.069
			5	2.2			
15-7PP	S-27205	Pier 2 (Span 3)	3	0.93	1.57	0.033	0.072
			5	2.2			
15-7PP	S-27206	Pier 1 (Span 1)	3	0.93	1.57	0.033	0.072
			5	2.2			
15-7PP	S-27206	Pier 1 (Span 2)	3	0.93	1.57	0.032	0.069
			5	2.2			
15-7PP	S-27206	Pier 2 (Span 2)	3	0.93	1.57	0.032	0.069
			5	2.2			
15-7PP	S-27206	Pier 2 (Span 3)	3	0.93	1.57	0.033	0.072
			5	2.2			
15-7PP	S-27207	Pier 1 (Span 1)	3	0.93	1.57	0.035	0.075
			5	2.2			
15-7PP	S-27207	Pier 1 (Span 2)	3	0.93	1.57	0.033	0.072
			5	2.2			
15-7PP	S-27207	Pier 2 (Span 2)	3	0.93	1.57	0.033	0.072
			5	2.2			
15-7PP	S-27207	Pier 2 (Span 3)	3	0.93	1.57	0.035	0.075
			5	2.2			

Table A4: PennDOT project 180-044 strength and cracking control calculations: Part 1

Project ID	S#
180-044	**26130**
W27x84,W24x68	
Girder Spacing (ft.)	7.79
# of girders	6
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	8.50
top cover (in)	2.00
bottom cover(in)	1.00
dneg (in)	6.19
dpos (in)	7.19
(+)M (kip/ft)	16.3
(+)M As (in^2/ft)	0.53
(-) M (kip/ft)	18.8
(-)M As (in^2/ft)	**0.73**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	71.4
(+) M equivalent Strip	77.4
dc (in)	2.31
h (in)	8.50
Mservice (kip/foot strip)	8.55
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.30
j	0.90
fs (ksi)	29.8
Υ_e	0.75
β_s	1.53
Required Spacing for Crack Control (S_{reqd}) (in)	6.88

Table A5: PennDOT project 180-044 strength and cracking control calculations: Part 2

Project ID	S#
180-044	**26163**
Eastbound	
Precast I-Beam	
24/48, 20/30	
Girder Spacing (ft.)	8.00
# of girders	6
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	7.50
top cover (in)	2.00
bottom cover(in)	1.00
dneg (in)	5.19
dpos (in)	6.19
(+)M (kip/ft)	16.6
(+)M As (in^2/ft)	0.64
(-) M (kip/ft)	19.0
(-)M As (in^2/ft)	**0.91**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	72.0
(+) M equivalent Strip	78.8
dc (in)	2.31
h (in)	7.50
Mservice (kip/foot strip)	8.66
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.33
j	0.89
fs (ksi)	36.3
Υ_e	0.75
β_s	1.64
Required Spacing for Crack Control (S_{reqd}) ***(in)***	4.22

Table A6: PennDOT project 180-044 strength and cracking control calculations: Part 3

Project ID	S#
180-044	**26163**
Westbound	
Precast I-Beam	
24/48, 20/30	
Girder Spacing (ft.)	6,67
# of girders	7
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	7.50
top cover (in)	2.00
bottom cover(in)	1.00
dneg (in)	5.19
dpos (in)	6.19
(+)M (kip/ft)	16.6
(+)M As (in^2/ft)	0.64
(-) M (kip/ft)	19.0
(-)M As (in^2/ft)	**0.91**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	68.0
(+) M equivalent Strip	70.0
dc (in)	2.31
h (in)	7.50
Mservice (kip/foot strip)	8.68
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.33
j	0.89
fs (ksi)	36.3
Υ_e	0.75
β_s	1.64
Required Spacing for Crack Control (S_{reqd}) (in)	4.20

Table A7: PennDOT project 180-044 strength and cracking control calculations: Part 4

Project ID	S#
180-044	**26129**
Westbound Pier 1	
Precast I-Beam	
24/48, 20/30, 24/60	
Girder Spacing (ft.)	6.00
# of girders	10
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	7.50
top cover (in)	1.00
bottom cover(in)	2.00
dneg (in)	6.19
dpos (in)	5.19
(+)M (kip/ft)	16.0
(+)M As (in^2/ft)	0.75
(-) M (kip/ft)	18.5
(-)M As (in^2/ft)	**0.71**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	66.0
(+) M equivalent Strip	65.6
dc (in)	1.31
h (in)	7.50
Mservice (kip/foot strip)	8.49
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.30
j	0.90
fs (ksi)	29.5
Υ_e	0.75
β_s	1.30
Required Spacing for Crack Control (S_{reqd}) (in)	11.01

Table A8: PennDOT project 180-044 strength and cracking control calculations: Part 5

Project ID	S#
180-044	**26129**
Westbound Pier 2	
Precast I-Beam	
24/48, 20/30, 24/60	
Girder Spacing (ft.)	5.50
# of girders	10
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	7.50
top cover (in)	1.00
bottom cover(in)	2.00
dneg (in)	6.19
dpos (in)	5.19
(+)M (kip/ft)	15.9
(+)M As (in^2/ft)	0.74
(-) M (kip/ft)	18.3
(-)M As (in^2/ft)	**0.71**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	64.5
(+) M equivalent Strip	62.3
dc (in)	1.31
h (in)	7.50
Mservice (kip/foot strip)	8.40
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.30
j	0.90
fs (ksi)	29.2
Υ_e	0.75
β_s	1.30
Required Spacing for Crack Control (S_{reqd}) (in)	11.16

Table A9: PennDOT project 180-044 strength and cracking control calculations: Part 6

Project ID	S#
180-044	**26129**
Eastbound Pier 1	
Precast I-Beam	
24/48, 20/30, 24/60	
Girder Spacing (ft.)	7.50
# of girders	8
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	7.50
top cover (in)	1.00
bottom cover(in)	2.00
dneg (in)	6.19
dpos (in)	5.19
(+)M (kip/ft)	15.9
(+)M As (in^2/ft)	0.75
(-) M (kip/ft)	18.4
(-)M As (in^2/ft)	**0.71**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	70.5
(+) M equivalent Strip	75.5
dc (in)	1.31
h (in)	7.50
Mservice (kip/foot strip)	8.45
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.30
j	0.90
fs (ksi)	29.4
Υ_e	0.75
β_s	1.30
Required Spacing for Crack Control (S_{reqd}) (in)	11.08

Table A10: PennDOT project 180-044 strength and cracking control calculations: Part 7

Project ID	S#
180-044	**26129**
Eastbound Pier 2	
Precast I-Beam	
24/48, 20/30, 24/60	
Girder Spacing (ft.)	7.50
# of girders	8
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	7.50
top cover (in)	1.00
bottom cover(in)	2.00
dneg (in)	6.19
dpos (in)	5.19
(+)M (kip/ft)	15.9
(+)M As (in^2/ft)	0.75
(-) M (kip/ft)	18.4
(-)M As (in^2/ft)	**0.71**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	70.5
(+) M equivalent Strip	75.5
dc (in)	1.31
h (in)	7.50
Mservice (kip/foot strip)	8.45
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.30
j	0.90
fs (ksi)	29.4
Υ_e	0.75
β_s	1.30
Required Spacing for Crack Control (S_{reqd}) (in)	11.08

Table A11: PennDOT project 180-044 strength and cracking control calculations: Part 1

Project ID	S#
180-058	**27091**
Westbound Pier 1	
Precast I-Beam	
24/54, 24/42	
Girder Spacing (ft.)	5.15
# of girders	9
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	7.50
top cover (in)	2.00
bottom cover(in)	1.00
dneg (in)	5.19
dpos (in)	6.19
(+)M (kip/ft)	14.1
(+)M As (in^2/ft)	0.53
(-) M (kip/ft)	14.3
(-)M As (in^2/ft)	**0.66**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	63.5
(+) M equivalent Strip	60.0
dc (in)	2.31
h (in)	7.50
Mservice (kip/foot strip)	6.54
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.33
j	0.89
fs (ksi)	27.4
Υ_e	0.75
β_s	1.64
Required Spacing for Crack Control (S_{reqd}) (in)	7.08

Table A12: PennDOT project 180-044 strength and cracking control calculations: Part 2

Project ID	S#
180-058	**27091**
Westbound Pier 2	
Precast I-Beam	
24/54, 24/42	
Girder Spacing (ft.)	5.42
# of girders	9
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	7.50
top cover (in)	2.00
bottom cover(in)	1.00
dneg (in)	5.19
dpos (in)	6.19
(+)M (kip/ft)	14.1
(+)M As (in^2/ft)	0.54
(-) M (kip/ft)	14.3
(-)M As (in^2/ft)	**0.66**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	64.3
(+) M equivalent Strip	61.8
dc (in)	2.31
h (in)	7.50
Mservice (kip/foot strip)	6.55
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.33
j	0.89
fs (ksi)	27.4
Υ_e	0.75
β_s	1.64
Required Spacing for Crack Control (S_{reqd}) (in)	7.07

Table A13: PennDOT project 180-044 strength and cracking control calculations: Part 3

Project ID	S#
180-058	**27091**
Westbound Pier 3	
Precast I-Beam	
24/54, 24/42	
Girder Spacing (ft.)	5.67
# of girders	9
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	7.50
top cover (in)	2.00
bottom cover(in)	1.00
dneg (in)	5.19
dpos (in)	6.19
(+)M (kip/ft)	14.1
(+)M As (in^2/ft)	0.53
(-) M (kip/ft)	14.3
(-)M As (in^2/ft)	**0.66**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	66.0
(+) M equivalent Strip	65.6
dc (in)	2.31
h (in)	7.50
Mservice (kip/foot strip)	6.55
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.33
j	0.89
fs (ksi)	27.4
Υ_e	0.75
β_s	1.64
Required Spacing for Crack Control (S_{reqd}) (in)	7.07

Table A14: PennDOT project 180-044 strength and cracking control calculations: Part 4

Project ID	S#
180-058	**27091**
Eastbound Pier 1	
Precast I-Beam	
24/54, 24/42	
Girder Spacing (ft.)	6.08
# of girders	8
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	7.50
top cover (in)	2.00
bottom cover(in)	1.00
dneg (in)	5.19
dpos (in)	6.19
(+)M (kip/ft)	14.1
(+)M As (in^2/ft)	0.54
(-) M (kip/ft)	14.3
(-)M As (in^2/ft)	**0.66**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	66.2
(+) M equivalent Strip	66.1
dc (in)	2.31
h (in)	7.50
Mservice (kip/foot strip)	6.57
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.33
j	0.89
fs (ksi)	27.5
Υ_e	0.75
β_s	1.64
Required Spacing for Crack Control (S_{reqd}) (in)	7.03

Table A15: PennDOT project 180-044 strength and cracking control calculations: Part 5

Project ID	S#
180-058	**27091**
Eastbound Pier 2	
Precast I-Beam	
24/54, 24/42	
Girder Spacing (ft.)	6.13
# of girders	8
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	7.50
top cover (in)	2.00
bottom cover(in)	1.00
dneg (in)	5.19
dpos (in)	6.19
(+)M (kip/ft)	14.1
(+)M As (in^2/ft)	0.54
(-) M (kip/ft)	14.3
(-)M As (in^2/ft)	**0.66**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	66.4
(+) M equivalent Strip	66.5
dc (in)	2.31
h (in)	7.50
Mservice (kip/foot strip)	6.56
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.33
j	0.89
fs (ksi)	27.5
Υ_e	0.75
β_s	1.64
Required Spacing for Crack Control (S_{reqd}) (in)	7.05

Table A16: PennDOT project 180-044 strength and cracking control calculations: Part 6

Project ID	S#
180-058	**27091**
Eastbound Pier 3	
Precast I-Beam	
24/54, 24/42	
Girder Spacing (ft.)	6.33
# of girders	8
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	7.50
top cover (in)	2.00
bottom cover(in)	1.00
dneg (in)	5.19
dpos (in)	6.19
(+)M (kip/ft)	14.1
(+)M As (in^2/ft)	0.54
(-) M (kip/ft)	14.3
(-)M As (in^2/ft)	**0.66**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	67.0
(+) M equivalent Strip	67.8
dc (in)	2.31
h (in)	7.50
Mservice (kip/foot strip)	6.57
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.33
j	0.89
fs (ksi)	27.5
Υ_e	0.75
β_s	1.64
Required Spacing for Crack Control (S_{reqd}) (in)	7.03

Table A17: PennDOT project 180-044 strength and cracking control calculations: Part 7

Project ID	S#
180-058	**27329**
Precast Box	
48"x42"	
Girder Spacing (ft.)	8.00
# of girders	7
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	7.50
top cover (in)	2.00
bottom cover(in)	1.00
dneg (in)	5.19
dpos (in)	6.19
(+)M (kip/ft)	19.3
(+)M As (in^2/ft)	0.75
(-) M (kip/ft)	21.0
(-)M As (in^2/ft)	**1.02**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	72.0
(+) M equivalent Strip	78.8
dc (in)	2.31
h (in)	7.50
Mservice (kip/foot strip)	9.63
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.33
j	0.89
fs (ksi)	40.3
Υ_e	0.75
β_s	1.64
*Required Spacing for Crack Control (S_{reqd}) **(in)***	3.33

Table A18: PennDOT project 180-044 strength and cracking control calculations: Part 8

Project ID	S#
180-058	**27165**
Westbound	
Precast Box	
48"x48"	
Girder Spacing (ft.)	7.83
# of girders	6
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	7.50
top cover (in)	2.00
bottom cover(in)	1.00
dneg (in)	5.19
dpos (in)	6.19
(+)M (kip/ft)	16.3
(+)M As (in^2/ft)	0.62
(-) M (kip/ft)	18.7
(-)M As (in^2/ft)	**0.90**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	71.5
(+) M equivalent Strip	77.7
dc (in)	2.31
h (in)	7.50
Mservice (kip/foot strip)	8.61
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.33
j	0.89
fs (ksi)	36.0
Υ_e	0.75
β_s	1.64
Required Spacing for Crack Control (S_{reqd}) (in)	4.27

Table A19: PennDOT project 180-044 strength and cracking control calculations: Part 9

Project ID	S#
180-058	**27165**
Eastbound	
Precast Box	
48"x48"	
Girder Spacing (ft.)	7.08
# of girders	8
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	7.50
top cover (in)	2.00
bottom cover(in)	1.00
dneg (in)	5.19
dpos (in)	6.19
(+)M (kip/ft)	16.2
(+)M As (in^2/ft)	0.62
(-) M (kip/ft)	18.7
(-)M As (in^2/ft)	**0.89**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	69.2
(+) M equivalent Strip	72.7
dc (in)	2.31
h (in)	7.50
Mservice (kip/foot strip)	8.59
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.33
j	0.89
fs (ksi)	36.0
Υ_e	0.75
β_s	1.64
Required Spacing for Crack Control (S_{reqd}) (in)	4.30

Table A20: PennDOT project 180-044 strength and cracking control calculations: Part 10

Project ID	S#
180-058	**27187**
Westbound Pier 1	
Precast I-Beams	
20/33, 26/63	
Girder Spacing (ft.)	6.92
# of girders	7
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	8.00
top cover (in)	2.00
bottom cover(in)	1.00
dneg (in)	5.69
dpos (in)	6.69
(+)M (kip/ft)	15.3
(+)M As (in^2/ft)	0.53
(-) M (kip/ft)	17.5
(-)M As (in^2/ft)	**0.74**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	69.0
(+) M equivalent Strip	72.2
dc (in)	2.31
h (in)	8.00
Mservice (kip/foot strip)	8.05
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.32
j	0.89
fs (ksi)	30.6
Υ_e	0.75
β_s	1.58
Required Spacing for Crack Control (S_{reqd}) (in)	6.22

Table A21: PennDOT project 180-044 strength and cracking control calculations: Part 11

Project ID	S#
180-058	**27187**
Westbound Pier 2	
Precast I-Beams	
20/33, 26/63	
Girder Spacing (ft.)	7.00
# of girders	7
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	8.00
top cover (in)	2.00
bottom cover(in)	1.00
dneg (in)	5.69
dpos (in)	6.69
(+)M (kip/ft)	15.2
(+)M As (in^2/ft)	0.53
(-) M (kip/ft)	17.5
(-)M As (in^2/ft)	**0.74**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	69.0
(+) M equivalent Strip	72.2
dc (in)	2.31
h (in)	8.00
Mservice (kip/foot strip)	8.05
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.32
j	0.89
fs (ksi)	30.6
Υ_e	0.75
β_s	1.58
Required Spacing for Crack Control (S_{reqd}) (in)	6.22

Table A22: PennDOT project 180-044 strength and cracking control calculations: Part 12

Project ID	S#
180-058	**27187**
Eastbound Pier 1	
Precast I-Beams	
20/33, 26/63	
Girder Spacing (ft.)	6.00
# of girders	10
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	8.00
top cover (in)	2.00
bottom cover(in)	1.00
dneg (in)	5.69
dpos (in)	6.69
(+)M (kip/ft)	15.2
(+)M As (in^2/ft)	0.53
(-) M (kip/ft)	17.5
(-)M As (in^2/ft)	**0.74**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	66.0
(+) M equivalent Strip	65.6
dc (in)	2.31
h (in)	8.00
Mservice (kip/foot strip)	8.02
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.32
j	0.89
fs (ksi)	30.5
Υ_e	0.75
β_s	1.58
Required Spacing for Crack Control (S_{reqd}) (in)	6.26

Table A23: PennDOT project 180-044 strength and cracking control calculations: Part 13

Project ID	S#
180-058	**27187**
Eastbound Pier 2	
Precast I-Beams	
20/33, 26/63	
Girder Spacing (ft.)	6.17
# of girders	10
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	8.00
top cover (in)	2.00
bottom cover(in)	1.00
dneg (in)	5.69
dpos (in)	6.69
(+)M (kip/ft)	15.2
(+)M As (in^2/ft)	0.53
(-) M (kip/ft)	17.5
(-)M As (in^2/ft)	**0.74**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	66.5
(+) M equivalent Strip	66.7
dc (in)	2.31
h (in)	8.00
Mservice (kip/foot strip)	8.02
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.32
j	0.89
fs (ksi)	30.5
Υ_e	0.75
β_s	1.58
Required Spacing for Crack Control (S_{reqd}) (in)	6.26

Table A24: PennDOT project 180-044 strength and cracking control calculations: Part 1

Project ID	S#
15-7PP	**27206**
Precast I-Beams	
Girder Spacing (ft.)	7.25
# of girders	7
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	8.75
top cover (in)	3.00
bottom cover(in)	1.00
dneg (in)	5.44
dpos (in)	7.44
(+)M (kip/ft)	15.5
(+)M As (in^2/ft)	0.48
(-) M (kip/ft)	18.0
(-)M As (in^2/ft)	**0.81**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	69.8
(+) M equivalent Strip	73.9
dc (in)	3.31
h (in)	8.75
Mservice (kip/foot strip)	8.26
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.32
j	0.89
fs (ksi)	32.9
Υ_e	0.75
β_s	1.87
*Required Spacing for Crack Control (S_{reqd}) **(in)***	1.90

Table A25: PennDOT project 180-044 strength and cracking control calculations: Part 2

Project ID	S#
15-7PP	**27207**
Precast I-Beams	
Girder Spacing (ft.)	8.50
# of girders	6
fy (ksi)	60
f'c (ksi)	5
bars	#5
bar area (in^2)	0.31
bar diameter (in)	0.63
deck depth (in)	9.25
top cover (in)	3.00
bottom cover(in)	2.00
dneg (in)	5.94
dpos (in)	6.94
(+)M (kip/ft)	17.6
(+)M As (in^2/ft)	0.59
(-) M (kip/ft)	19.6
(-)M As (in^2/ft)	**0.80**
As provided	**0.62**
Check for Crack Control	
Modular Ratio (n)	8.00
(-) M equivalent Strip	73.5
(+) M equivalent Strip	82.1
dc (in)	3.31
h (in)	9.25
Mservice (kip/foot strip)	9.01
b (in)	6.00
Reinforcement Ratio (ρ)	0.01
k	0.31
j	0.90
fs (ksi)	32.7
Υ_e	0.75
β_s	1.80
Required Spacing for Crack Control (S_{reqd}) (in)	2.30

Appendix B

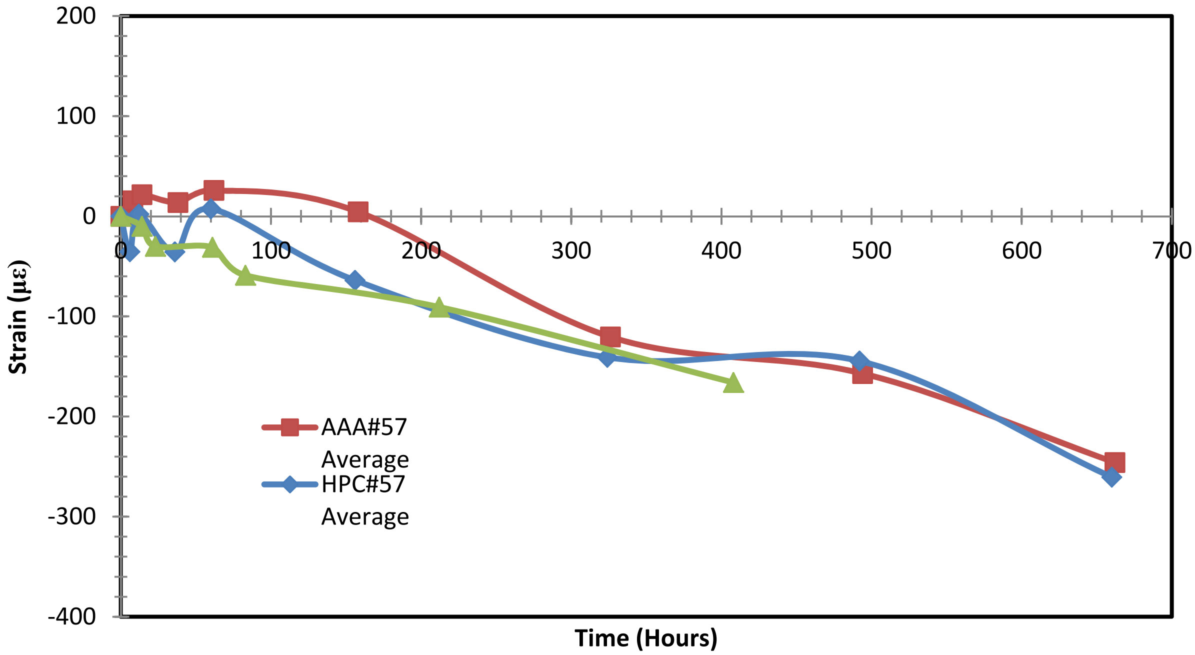

Figure B1: Average of four autogenous strain specimens over 28 days

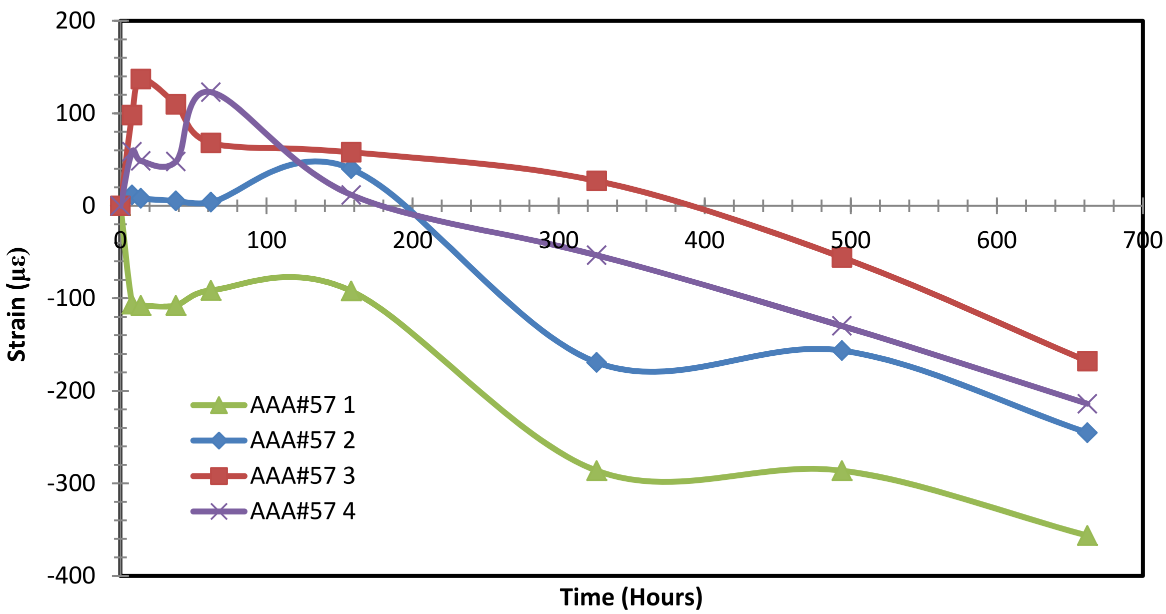

Figure B2: AAA#57 cement paste autogenous strain specimens over 28 days

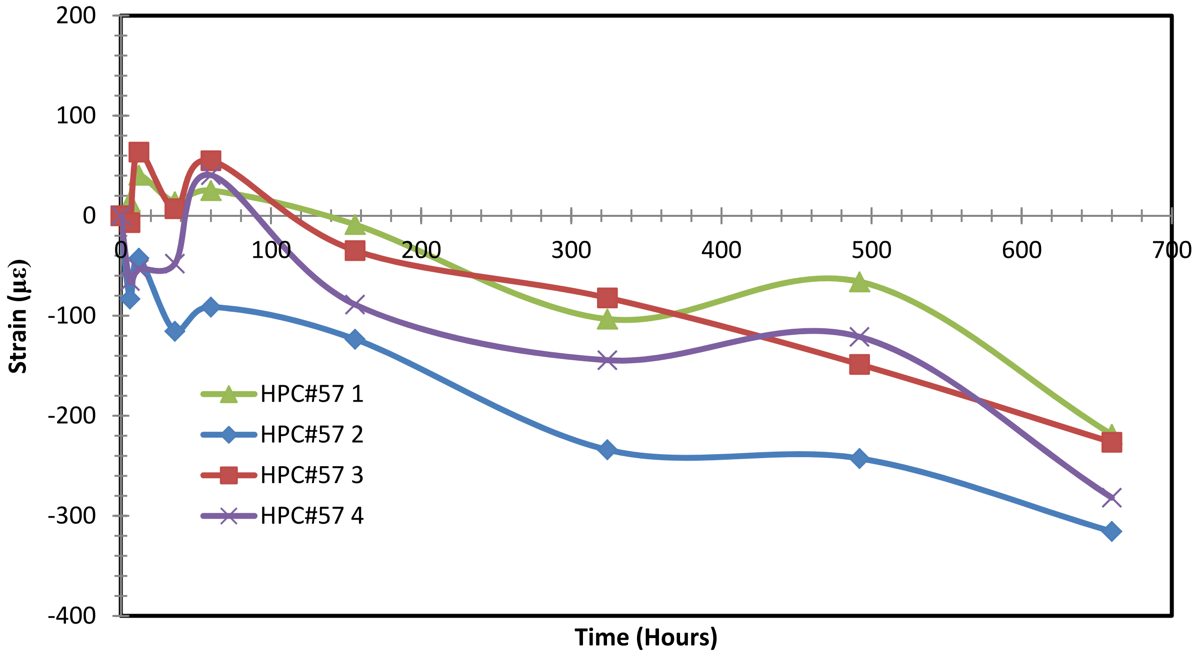

Figure B3: HPC#57 cement paste autogenous strain specimens over 28 days

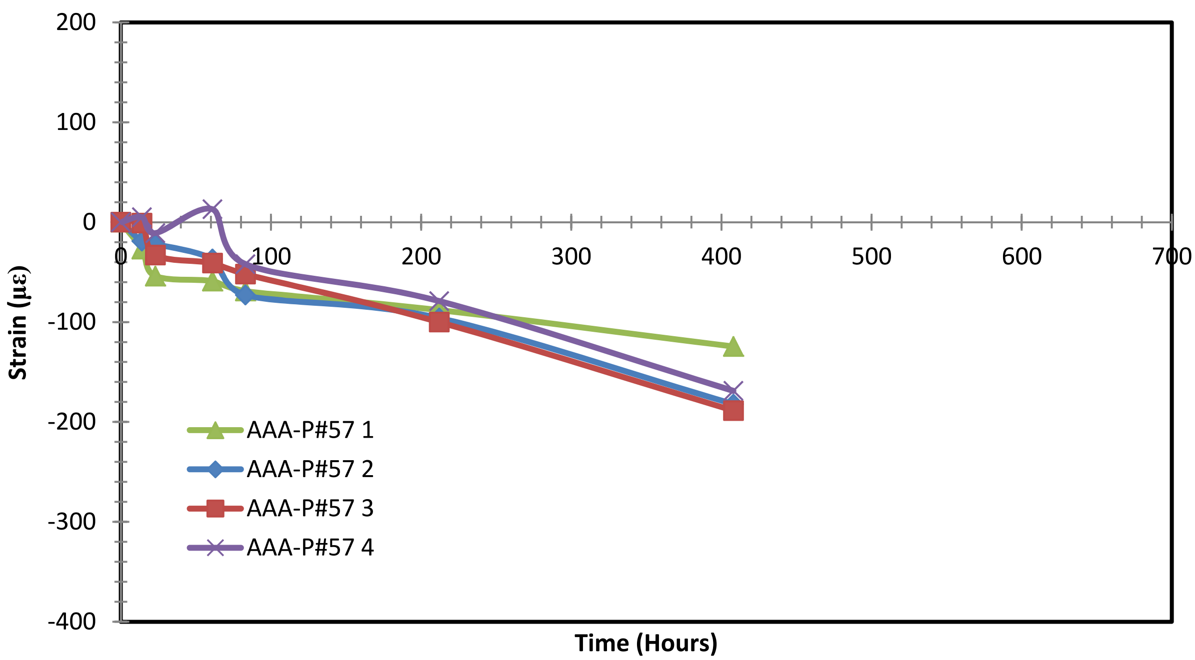

Figure B4: AAA-P#57 cement paste autogenous strain specimens over 20 days

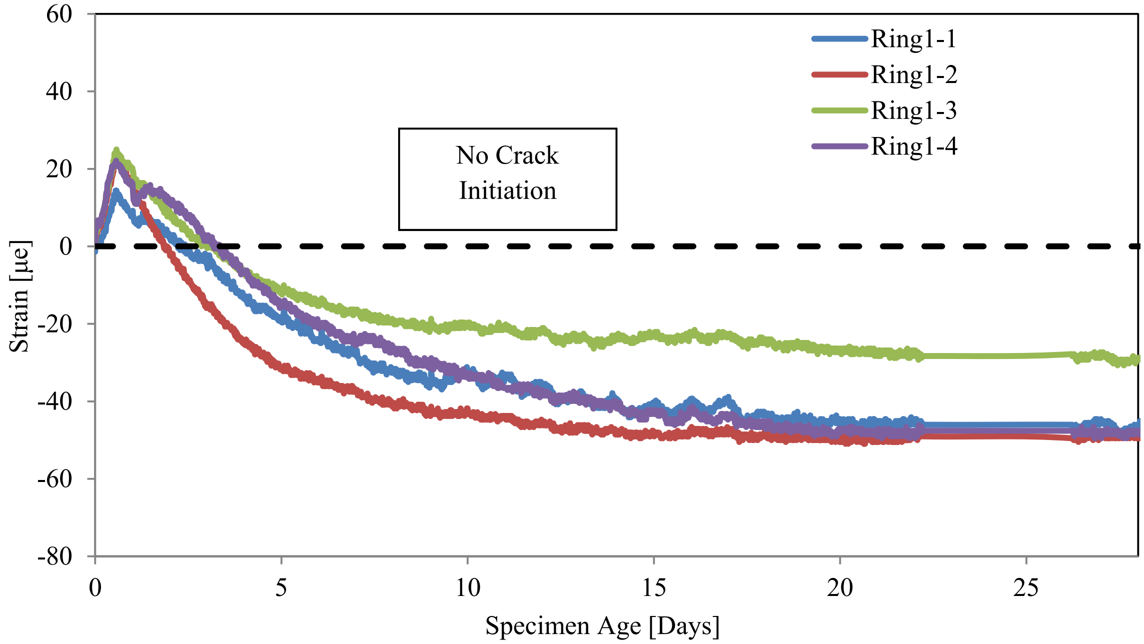

Figure B5: AAA#57 Ring 1 restrained shrinkage strain

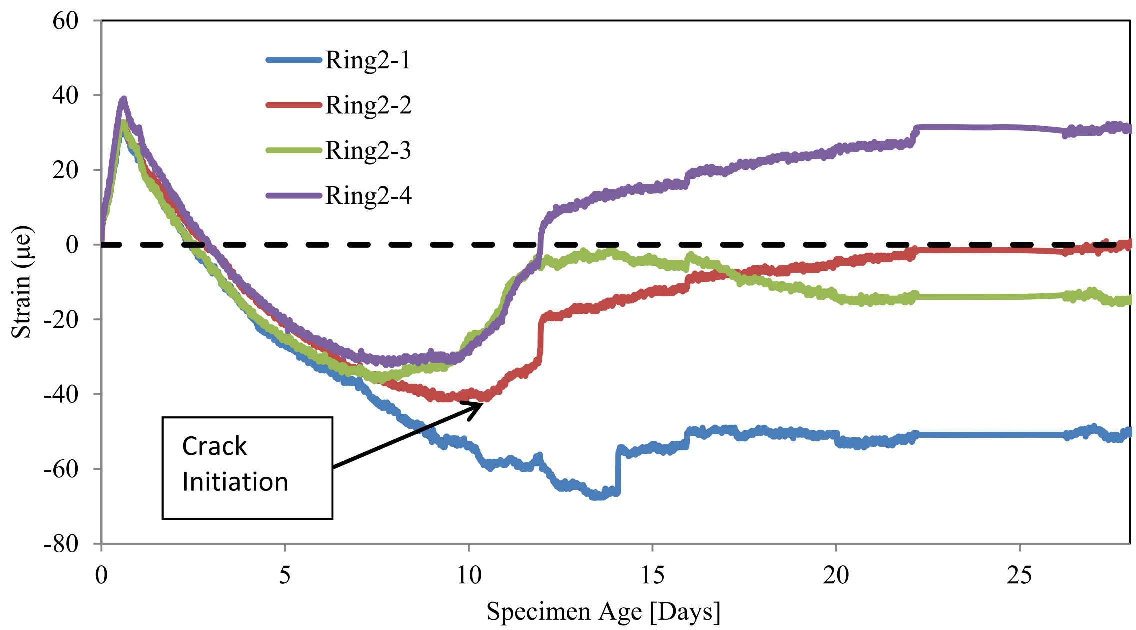

Figure B6: AAA#57 Ring 2 restrained shrinkage strain

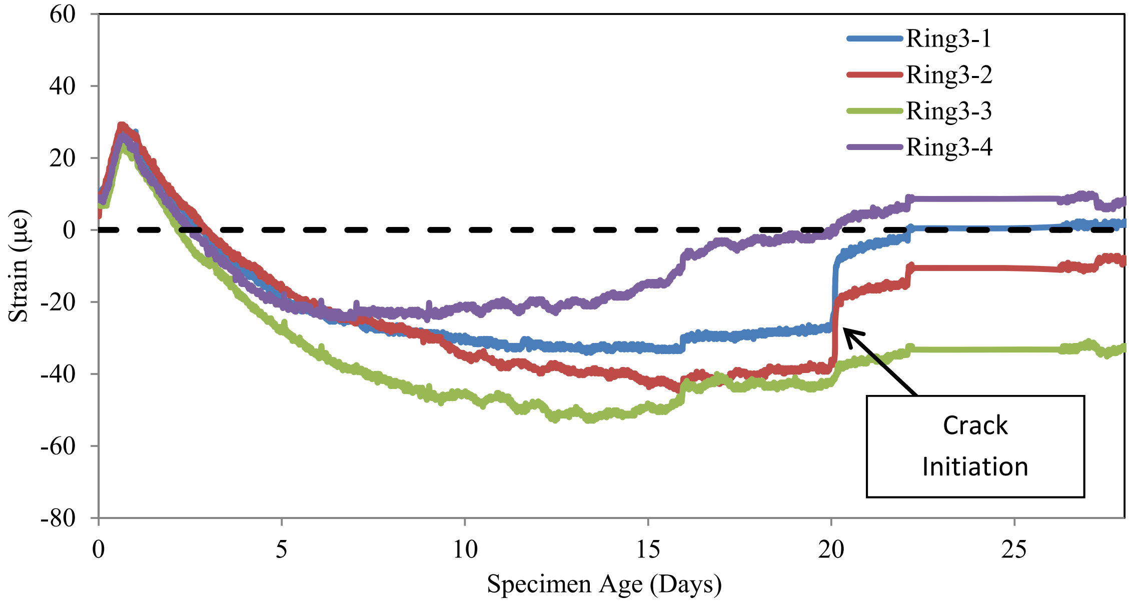

Figure B7: AAA#57 Ring 3 restrained shrinkage strain

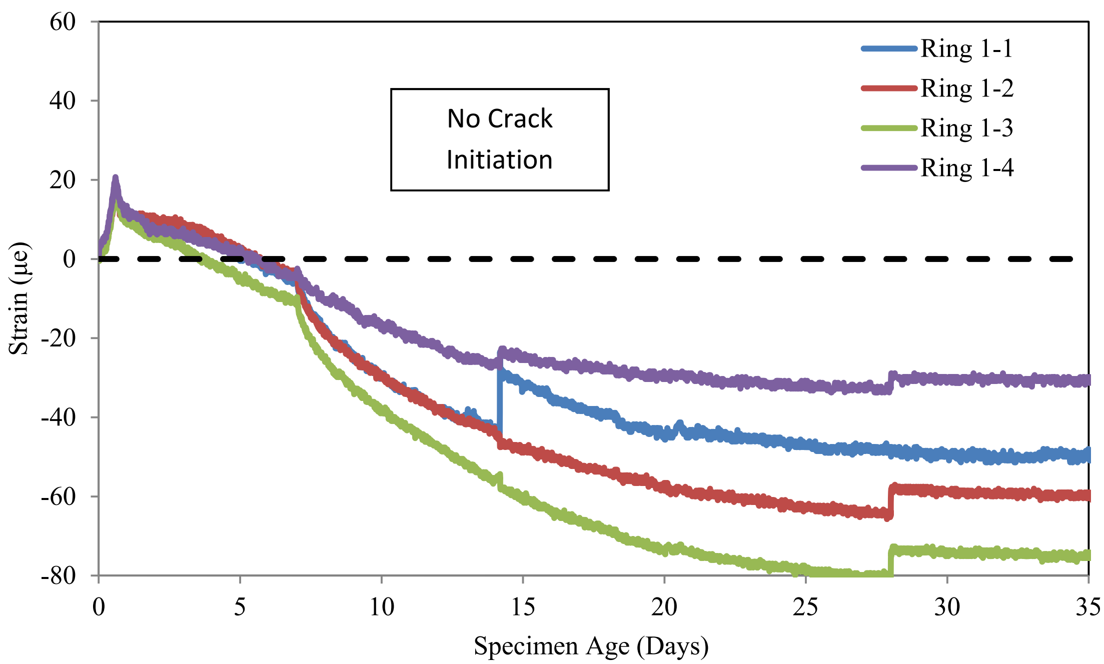

Figure B8: HPC#57 Ring 1 restrained shrinkage strain

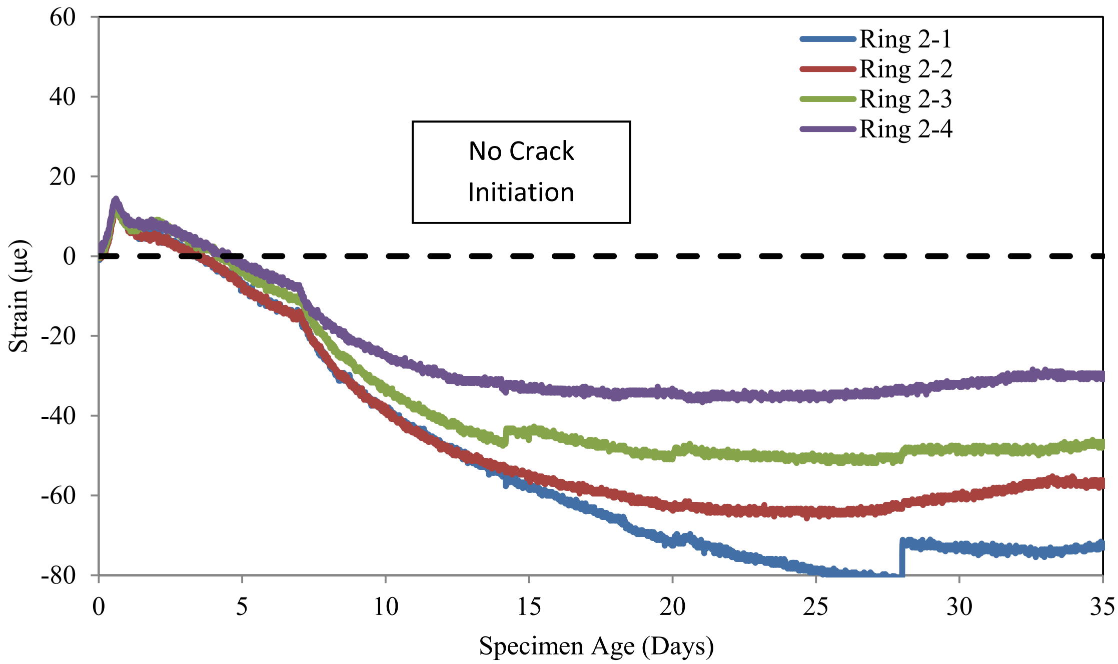

Figure B9: HPC#57 Ring 2 restrained shrinkage strain

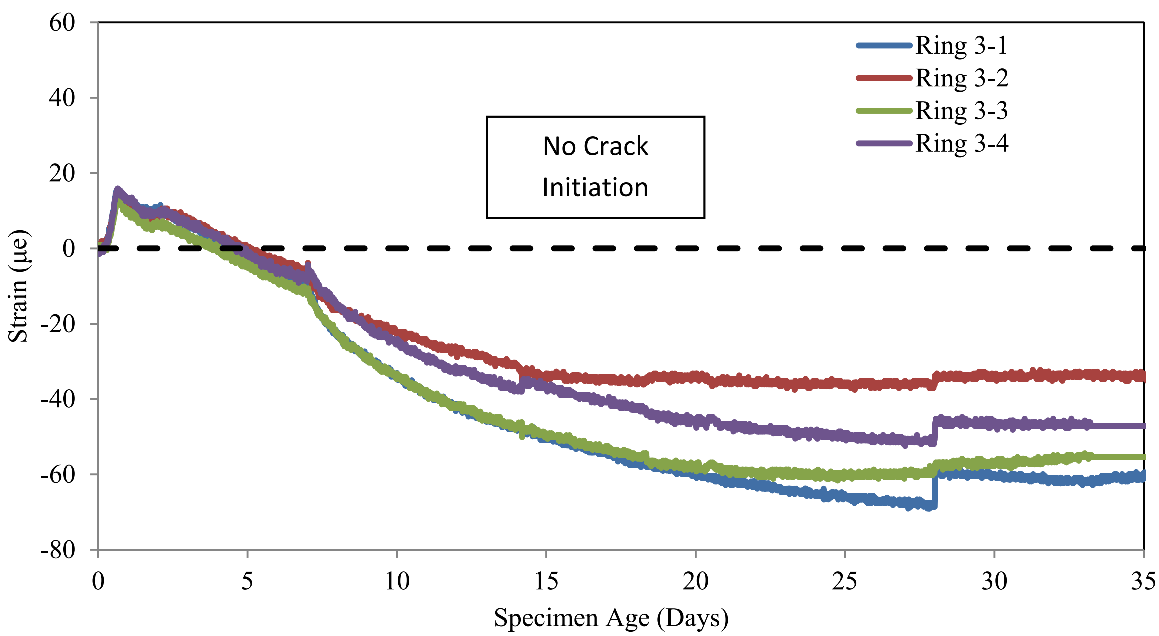

Figure B10: HPC#57 Ring 3 restrained shrinkage strain