HIGH PLASTIC CONCRETE TEMPERATURE SPECIFICATIONS FOR PAVING MIXTURES

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16. Abstract

This report documents a study performed for the Illinois Department of Transportation (IDOT) regarding concrete roadway construction in hot weather. The main objective in this project is to develop improved specifications and procedures with respect to monitoring and maintaining plastic concrete temperatures to assure near and long-term concrete quality in the state of Illinois. An additional objective is to develop a better understanding of the effects of higher temperatures on fresh and hardened concrete properties of roadway concrete and to document any new findings.

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EXECUTIVE SUMMARY

Portland cement concrete can develop undesirable characteristics when the material is subjected to high temperatures during mixing, transporting, casting, finishing, and curing. The state of Illinois raised the accepted threshold placement temperature value from 90°F to 96°F (32.2°C to 35.6°C) in the Standard Specifications for Road and Bridge Construction (Adopted July 1, 1994) for pavement. Since that time, instances of poor concrete quality have been noted among Illinois Department of Transportation (IDOT) engineers. The current IDOT specification (Section 1020.14, presented in Appendix A) states the maximum concrete temperature before placement should be less than or equal to 90°F (32.2°C) and the maximum concrete temperature as placed should not exceed 96°F (35.6°C). This report documents a study performed by the University of Illinois at Urbana Champaign for IDOT regarding its concrete roadway material specification and construction practices in hot weather. The main objective of this project was to develop improved specifications and procedures with respect to monitoring and maintaining plastic concrete temperatures to assure near and long-term concrete quality in the state of Illinois. An additional objective was to better understand the effects of higher temperatures on fresh and hardened concrete properties for materials available in Illinois and to document any new findings.

The report first reviews the current state of understanding and practice for hot weather concrete placement. A review of concrete ring shrinkage tests was conducted to assess its applicability for this project. This review is documented in Appendix E. A survey of states' practices for hot weather concreting was carried out by the research team and the detailed responses are presented in Appendices C and D. The survey results indicated that most states specify a maximum allowable plastic concrete temperature of 90°F (32.2°C) or lower, although several states including nearly all of the neighboring states to Illinois, do not specify a maximum allowable temperature for plastic concrete. Illinois is the only state that specifies two separate maximum plastic concrete temperatures, one at delivery before placement (90°F or 32.2°C) and one as placed (96°F or 35.6°C). The results also suggest that the states that have a maximum allowable limit of 90°F (32.2°C) or lower control the hot weather concreting problem fairly well, as these states also report low occurrence of temperature-related concrete distresses.

The investigating team conducted three visits to roadway sites in Illinois with suspected hot weather concrete problems. The three site locations visited were in Chicago, Peoria (three projects sites), and Collinsville, Illinois. The concrete at the sites exhibited some combination of excessive cracking, rapid slump loss, early setting time, and poor finishability. The common feature in these three sites is that concrete temperatures were high, either very close to or above 90°F (32.2°C). Core samples were obtained from the Peoria site locations for the purpose of performing a series of laboratory tests. The field samples were analyzed by standard resonance frequency analysis, the standard test for density, absorption, and voids, i.e. the "boil" test. Based on a 95 percent statistical confidence level, the test results obtained from the field concrete samples show no meaningful differences between hot and cooler temperature castings in terms of both dynamic modulus and permeable void volume data. Furthermore, statistically significant differences between the top and mid sections of the field core samples, owing to the temperature variation with depth of the plastic concrete, are not seen. Based on these data from the test site samples, we conclude that the distributed damage state and permeable pore structure of concrete at the micro- and meso-scales is not significantly affected by

concrete plastic temperatures within this sample set. Thus, the overall extent of this problem and long-term performance implication in Illinois is not clear.

To investigate the effects of mineral and chemical admixtures on the performance of fresh concrete at higher temperatures, a series of concrete mixtures were subjected to the high temperature mixing procedure developed by Florida Department of Transportation (FDOT). Air content and slump were measured before and after the 90-minute mixing procedure as well as compressive and splitting tensile strengths at various ages. All tested concrete mixtures with mineral (class C fly ash and ground granulated blast-furnace slag) and chemical (air-entraining admixture, superplasticizer, and retarder) admixtures met the 14-day minimum strength requirement for "PV" concrete. The results indicate that addition of retarding admixture is critical for minimizing slump loss and entrained air loss and extending placement and finishing time of concrete mixtures exposed to extended high temperatures during mixing, placement, and finishing. Together with retarding admixture, high percentage replacement of cement with ground granulated blast-furnace slag significantly reduced the concrete slump loss over the mixing time for the material combinations tested. However, retarding admixture, cement replacement with fly ash (Class C) did not notably affect the slump loss rate of concrete mixtures exposed to extended high temperature mixing.

Based on the results presented in the report, there does not appear to be systematic evidence of frequent cracking problems in Illinois concrete projects related to high temperature placement. However, there appears to be occasional placement, consolidating, and finishing issues with concrete when concrete temperatures approach or exceed 90°F (32.2°C). The authors recommend a change in specification language to reduce uncertainty and also encourage paving contractors to innovate and use improved cementitious materials at times when air and concrete temperatures are expected to exceed 90°F (32.2°C). In particular, we recommend that the current dual-valued temperature specification be replaced with one value of specified temperature: as delivered at 90°F (32.2°C). However to give flexibility to contractors, higher delivered temperatures, up to 96°F (35.6°C), could be allowed if the proposed mixtures meet existing IDOT performance targets for air, slump, and strength when using the FDOT high temperature mixing procedure. Furthermore, the authors recommend maintaining the current recommendation for addition of retarder. Finally, the authors recommend the removal of one statement from the specifications: that retarder requirement be waived if fly ash is added. The authors provide specific wording changes to the current specification.

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CHAPTER 1 INTRODUCTION

Portland cement concrete can develop undesirable characteristics when the material exhibits high plastic temperatures while it is being mixed, transported, cast, finished, and cured during hot weather. High plastic concrete temperatures affect important properties of the plastic mixture: increased water demand of the mixture, increased slump loss, reduction in setting times, increased tendency for plastic shrinkage cracking, difficulty in finishing, and reduced control of entrained air content. High mixture temperatures also affect important properties of the hardened concrete such as decreased ultimate strength, increased tendency for moisture and thermal shrinkage cracks, decreased material durability, and decreased uniformity of surface appearance (Samarin et al. 1982, Schindler and McCullough 2002).

Suitable precautions must be carried out in situations where high temperatures exist in order to achieve uniformly good concrete quality that will perform adequately in the plastic and hardened states. An important precaution is to ensure that the plastic concrete temperature be kept suitably below some defined threshold temperature. For many climates and construction conditions in the United States, that stipulated threshold material temperature value is 90°F (32.2°C). However, this value can be higher or lower than 90°F (32.2°C) depending on regional factors such as climatic conditions, concrete material constituents, construction process, and geometry of the structure. For example, the state of Illinois raised the accepted threshold placement temperature value from 90 to 96°F (32.2 to 35.6°C) in the Standard Specifications for Road and Bridge Construction (Adopted July 1, 1994) for pavement. Since that time, cases of poor concrete quality have been discussed among IDOT engineers. The current IDOT specification (Section 1020.14) states the maximum concrete temperature before placement should be less than or equal to 90°F (32.2°C) and the maximum concrete temperature as placed should not exceed 96°F (35.6°C). ACI 305.1-06 recommends maintaining concrete temperatures below 95°F (35°C) and stresses the importance of carefully monitoring conditions to minimize evaporation, especially until proper curing methods have been put in place. ACI also suggests several methods to reduce the temperature of concrete, including "shading aggregate stockpiles, sprinkling water on coarse aggregate stockpiles, using chilled water for concrete production, substituting chipped or shaved ice for portions of the mixing water, and cooling concrete materials using liquid nitrogen" (ACI 2006).

This report documents a study performed by the University of Illinois at Urbana Champaign (UIUC) for the Illinois Department of Transportation (IDOT) regarding concrete roadway construction in hot weather. The main objective in this project is to develop improved specifications and procedures with respect to monitoring and maintaining plastic concrete temperatures to assure near and long-term concrete quality in the state of Illinois. An additional objective is to develop a better understanding of the effects of higher temperatures on fresh and hardened concrete properties of roadway concrete and to document any new findings.

CHAPTER 2 CURRENT STATE OF UNDERSTANDING AND PRACTICE

2.1 REVIEW OF RECENT LITERATURE

The development of the concrete temperature after placement is a complex situation. It is mainly affected by the temperature of the concrete at placement, the curing temperature, the type and quantity of the mineral admixtures, the solar radiation intensity, and the boundary conditions of the pavement. Concrete is a poor conductor of heat, and the rate of heat evolution due to the hydration process is therefore much greater than the rate of heat dissipation. Consequently, the temperature inside the concrete rises during early hydration stages. This increase in the internal temperature can be further aggravated by the higher temperatures at mixing and placement, which increases the rise in concrete temperature.

Modern Portland cements, characterized by higher tri-calcium silicate (C_3S) content and fineness, hydrate more rapidly and therefore develop high early strength but also high heats of hydration. High internal temperature can result in early-age thermal cracking unless actions are taken to address the issue. Concrete placement under hot weather conditions further aggravates the problem. High concrete temperatures increase the rate of hydration, thermal stresses, the tendency for drying shrinkage cracking, and permeability. Higher initial concrete temperatures increase the early strength gain but decrease the long-term concrete strength and durability [Neville et al [1996]; Schindler et al [1813]. This is confirmed by the SHRP-C-321 study [A Guide to Evaluate et al [1992], which reported that the effects of temperature and moisture early in the life of concrete strongly influence early strength development and long-term durability. Other research findings also concluded that the concrete temperature development during the first 24 to 72 hours after placement has a major impact on long-term pavement performance [McCullough,B.F.; Dossey, T.,et al. [1999].

Various methods have shown indirectly that insufficient time for diffusion of the hydration products and the large pores that form as a result are responsible for the reduction in strength of concretes cured at elevated temperatures. [Verbeck, G.J.; Helmuth R.H., et al. [1968] presented an explanation for the reduced long-term strength for concretes cured at high temperatures. They suggested that a higher initial temperature results in more than a proportional increase in the initial rate of hydration. Therefore, during the early stage of curing, when there is rapid strength development, the strength of concrete cured at the high temperature is greater than that of concrete cured at the lower temperature. However, with rapid hydration, hydration products do not have time to become uniformly distributed within the pores of the hardening paste. In addition, "shells" made up of low-permeability hydration products build up around the cement grains. The non-uniform distribution of hydration products leads to more large pores, which affect the long-term strength, and the shell impedes hydration of the un-reacted portion of the grains at later ages [Kjellsen, K.O; Detwiler, R.J. Gjorv, O.E, et al [1991].

2.2 REVIEW OF MITIGATION OF EFFECTS OF HIGH TEMPERATURES

Suitable precautions must be carried out in situations where high air temperatures exist in order to achieve uniform, good concrete quality that will perform adequately in the plastic and hardened states. An important precaution is ensuring that the temperature of the plastic concrete be kept below some defined threshold temperature. For many climates and construction conditions in the United States, that stipulated threshold material temperature value is 90°F (32.2°C). However, this value can be higher or lower than 90°F (32.2°C).

depending on regional factors such as climate zone, concrete material constituents, construction method, and geometry of the structure. For example, the Illinois Department of Transportation (IDOT) raised the accepted threshold placement temperature value from 90°F (32.2°C) to 96°F (35.6°C). in 1994 (IDOT 2007) for all concrete other than that used in structures. Since that time, IDOT engineers have reported cases of poor concrete quality due to excessive placement temperatures.

As an increase in air temperature at placement will also produce an accompanied increase in concrete temperature. A number of mitigation measures can be used to minimize the potential of experiencing problems associated with placement of concrete in hot weather, of which the following are only a few examples [ACI Committee et al [1999]; Mehta, PK. et al [2002]:

The use of concrete materials and proportions with satisfactory performance in place under hot weather conditions;

The use of cooled concrete, which can be achieved by using chilled mixing water, ice in the mixture, or liquid nitrogen to cool the mixing water or the concrete mixture or by cooling the coarse aggregate;

The use of a concrete consistency that allows rapid placement and effective consolidation at high temperatures;

Scheduling of placement activities during the times of the day or night when the weather conditions are favorable;

Minimization of the time to transport, place, consolidate, and finish the concrete

Protection of the concrete from moisture loss at all times during placement and during its curing period; and limit the maximum in-place concrete temperature during placement of concrete.

There has been much discussion and debate about whether a temperature limit needs to be applied to concrete produced in hot weather. One argument is that additional mixing water is needed for hot concrete, thus reducing the strength and quality for a given cement content. However, the production of cooled concrete can be expensive if ice or refrigeration must be used. [Mehta,P.K. et al [2002] suggests adjustments or changes in the mixture such as mineral admixtures [ACI Committee, et al [1999]; Samari, M, Popovics S.; Malhotra (TRR 924, pp 42-50], and [Ma et al. [2004], e.g., high volume fly ash mixtures, can usually be made to produce concrete of the required quality, in the 32 to 38 ºC (90 to 100 ºF) temperature range, and typically these changes will be more cost-effective than cooling the concrete. It has been well documented that the use of mineral admixtures such as fly ash or ground granulated blast furnace slag can significantly slow the rate of heat evolution [Ma et al [2004].

In modern paving operations, the use of mineral admixtures has become common practice, e.g., 15 to 20 percent cement replacement, and under certain conditions, increases in mineral admixture replacement of cement could mitigate some of the problems associated with placement in hot weather. A specification limiting concrete temperature at placement might be appropriate for some conditions but unnecessary in others. The limits selected by most states were likely chosen on the basis of mixture designs that contained little to no mineral or chemical admixtures. The current prescriptive practice of high temperature concrete placement strategies is somewhat prohibitive and does not encourage measure to reduce mixture temperature during hot weather application. To encourage the use of less heat sensitive mixtures, specifications should differentiate between mixtures that have different heat evolution rates.

According to [Mehta, P.K. et al [2002], from theoretical considerations and field experience, under warm weather conditions, blended portland cements containing very high volumes of fly ash provide a better and more cost-effective approach for durable concrete structures. Although [Mehta, P.K. et al [2002] was addressing thermal cracking problems in

mass concrete castings, the principles may be applied to concrete pavements. These very high volumes of fly ash concretes are characterized by containing 50% or more cement replacement with fly ash by mass. A 50% substitution of cement with a pozzolan is expected to bring down the 7 day heat of hydration from about 70-80 to 45-50 cal/g [Mahta, P.K, Monteiro P.J., et al [1993]. Bilodeau, A,; Malhotra, V.M., et al [2000], reported that , in a large high volume fly ash concrete block, the temperature rise was limited to 95°F (35°C), compared to 149°F (65°C) reached in a block of the same size made of concrete containing Type I portland cement only.

Mehta,P,K.; Langely, W. et al [2000], reported the complete elimination of thermal cracking in a structure under hot weather conditions by the use of high volume fly ash concrete containing 57% fly ash by mass of the total cementitious materials. The only concern about these concretes is the strength behavior at early ages. According to [Manmohan, D.; Mehta, P.K., et al [2002], it is also possible to produce reasonable concrete strengths with high volumes of fly ash. Manmohan,D.; Mehta, P.K. et al [2002], reported the use of high volume fly ash concrete mixtures for the construction of a post-tensioned reinforced concrete. With the use of 50% Class F fly ash, the concrete reached 2900 psi (20 MPa) of compressive strength at 7 days.

CHAPTER 3 SURVEY OF STATES' PRACTICE

Here we examine the experiences of other states' practices for hot weathering concrete. The primary source of information is a survey that was sent out by the ICT research team to department of transportation contacts in all 50 states. Thirty-two states responded to the survey. In addition, 47 of the states' highway construction specifications were found and studied for insight into their hot weathering concrete policy. Several other sources of information are discussed here as well, such as a survey conducted several years ago by the South Carolina DOT, an article from *Concrete International* featuring Florida's unique hot weather policies, and an in-depth study of hot-weather policies in highway construction specifications from nearly every state.

3.1 SOUTH CAROLINA DOT SURVEY

In 2004, Merrill Zwanka of the South Carolina DOT sent a short survey to the rest of the state DOTs regarding Portland Cement Concrete (PCC) temperature requirements. The questions asked for the minimum and maximum allowable batching temperatures of PCC when placed in forms, as well as the maximum allowable temperature and temperature differential for mass concrete pours. Since these questions referred to structural concrete rather than pavement concrete and many states have separate requirements for structural and non-structural concrete, the South Carolina DOT survey was not very useful to this project.

However, Zwanka did receive responses from 30 states, as summarized below in Figures 1 and 2. Most of the states responded that they have a maximum allowable temperature of 90°F (32.2°C), though there were three states lower, at 70°F (21.1°C), 80°F (26.6°C), and 85°F (29.4°C), as well as four with higher allowable temperatures, at 93°F (38.3°C) and 100°F (37.8°C). Some states distinguished between temperature "before placement" or "in the forms," but most states did not give any indication of when this temperature is to be taken.
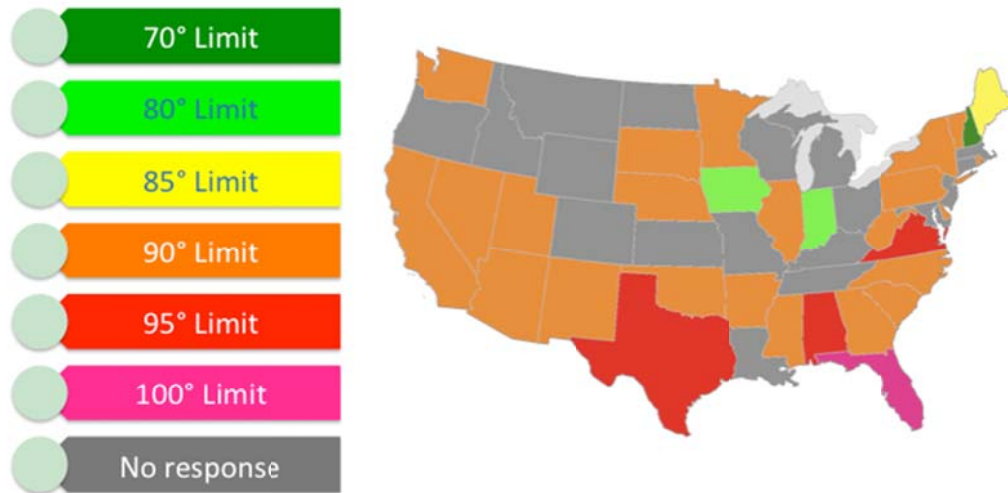
Figure 1. Maximum allowable concrete temperature (°F) map from South Carolina survey.
Note °C = (°F – 32) x 5/9.
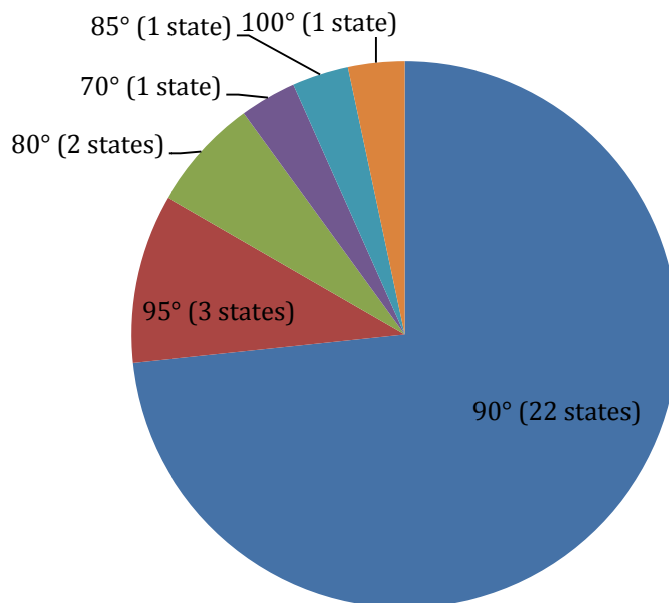
Figure 2. Summary of South Carolina DOT survey regarding maximum allowable
temperature (°F) for concrete placed in forms. Note °C = (°F – 32) x 5/9.

3.2 FLORIDA HOT WEATHER PRACTICE IN *CONCRETE INTERNATIONAL*

In May 2002, an article about Florida's hot weather concrete specifications appeared in *Concrete International,* the monthly magazine of the American Concrete Institute. The article explains that historically plastic concrete was limited to 80°F (26.6°C) or cooler, which understandably makes concrete construction very expensive, requiring ice, night pours, and shutdowns during hot weather. In 1989, Florida changed their specifications to allow a maximum allowable plastic temperature of 100°F (37.7°C), provided that the mixture can pass a series of laboratory tests prior to the casting of concrete on the actual project. If the

mixture does not pass the test, then the concrete is not approved for hot weather use, and work on the project must cease if the concrete temperatures exceed 85°F (30°C).

The laboratory test involves mixing the concrete in the laboratory, using ingredients heated to 90°F (32.2°C) to ensure that the plastic concrete reaches 94°F (34.4°C). The concrete is then mixed for 30 seconds every five minutes while held at 94°F (34.4°C) for 90 minutes. After 90 minutes, the concrete must meet the slump and air content requirements within a reasonable margin of error. Full detail of the current Florida hot weather concrete specification and hot weather mixture verification procedure is given in Appendix B.

In the field, the mixtures that are approved for hot weather must still be kept below 100°F (37.7°C) before final placement, and be placed within 45 minutes (for non-agitating trucks) or 60 minutes (for agitating trucks), measured from the time that the water is added to the truck. If water-reducing and retarding admixtures (ASTM type D or G) are used, the times are extended to 75 and 90 minutes, respectively.

Florida's specifications have been quite successful, and are popular with both DOT engineers and contractors. In the article, one contractor reports that he uses cooling methods (such as ice) on only 5% of his projects, and the cost of making trial mixtures is more than offset by the ability to pave year round and during the day.

3.3 STATE SPECIFICATIONS

Though the information gathered from the surveys (as discussed in the next section) contain more information about specific hot weather practices, state highway construction specifications were obtained separately from as many states as possible to fill in states which did not respond to the survey.

The specifications were also studied to determine when most states required that the temperature of concrete be taken. Most states are not specific about this, simply saying that the temperature must be below a certain temperature "before placement," "at the time of placement," or simply direct the contractor to "maintain the temperature at or below" the limiting temperature.
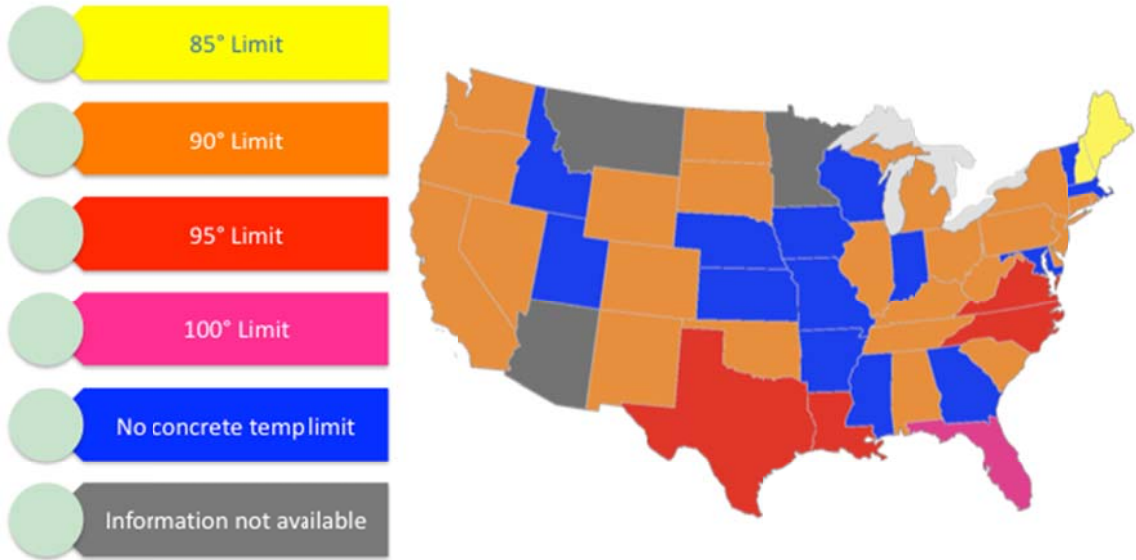
Figure 3. Results of UIUC investigation of maximum allowable concrete temperature (°F) from state specification documents
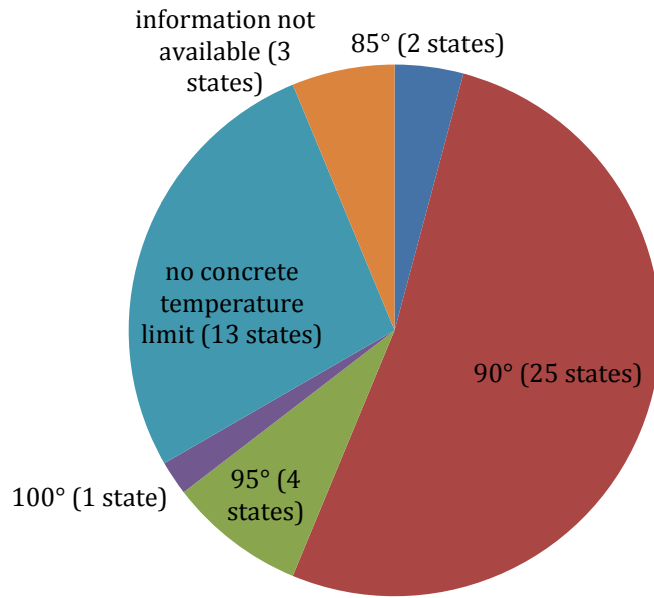
Figure 4. Maximum temperature (°F) results of UIUC investigation of state specifications. Note °C = (°F – 32) x 5/9.

The results of the specification study are shown in Figure 3 and Figure 4. Half the states require concrete to be at or below 90°F (32.2°C) before placement, but a quarter of the states have no limit or no mention of concreting in hot weather. Two states had a lower allowable temperature, 85°F (30°C), four states allow up to 95°F (35°C), and one state allows up to 100°F (37.7°C). Only the 48 contiguous states were studied with three of the states not having information available. Illinois is the only state that has a specification

featuring two separate plastic temperatures before placement and as placed. New Hampshire has an "as-delivered" plastic temperature and an "after placement" hardened concrete temperature of 100°F (37.7°C) to be maintained after the concrete is in place and curing.

3.4 UNIVERSITY OF ILLINOIS SURVEY

3.4.1 Survey Introduction and Summary

To obtain information on states' practices for hot weather concreting, a survey was sent by email to appropriate materials contacts in each state's department of transportation. The questions asked about the maximum allowable plastic concrete temperatures for pavements and structures, and asked if there was any deterioration that was attributed to paving in hot weather. Also, the survey asked what the state's standard practice for cooling concrete is, and if there were any mixture design related changes required to accommodate higher temperatures. Finally, the survey also asked about which party (i.e. state or contractor) absorbed lost time or extra costs due to high concrete temperatures. An example survey is attached in the Appendix C and all completed surveys in Appendix D of this report.

3.4.2 Basic Survey Results

As shown in Table 1, responses were obtained from 32 states, including states that are similar to Illinois, significantly warmer than Illinois, and significantly cooler than Illinois.

Table 1. Summary of Temperature Limit Responses

No Limit	8 states
85°F limit *	2 states
90°F limit	17 states
95°F limit	3 states
96°F limit	1 state
100°F limit	1 state
Note °C = (°F – 32) x 5/9	

The results of the UIUC survey are shown in Figure 5 and Figure 6. Over half the states require concrete to be at or below 90°F (32.2°C) before placement, but about a quarter of the states, including nearly all of the neighboring states to Illinois, have no limit or no mention of concreting in hot weather. Two states had a lower allowable temperature, 85°F (30°C), three states allow up to 95°F (35°C), and one state allows up to 100°F (37.7°C) (Florida). Only Illinois reported a maximum allowable temperature of 96°F (35.5°C) as placed, such that placement and finishing does not require any excess water or overworking the concrete surface. The maximum allowable delivered (before placement) temperature for Illinois is 90°F (32.2°C).
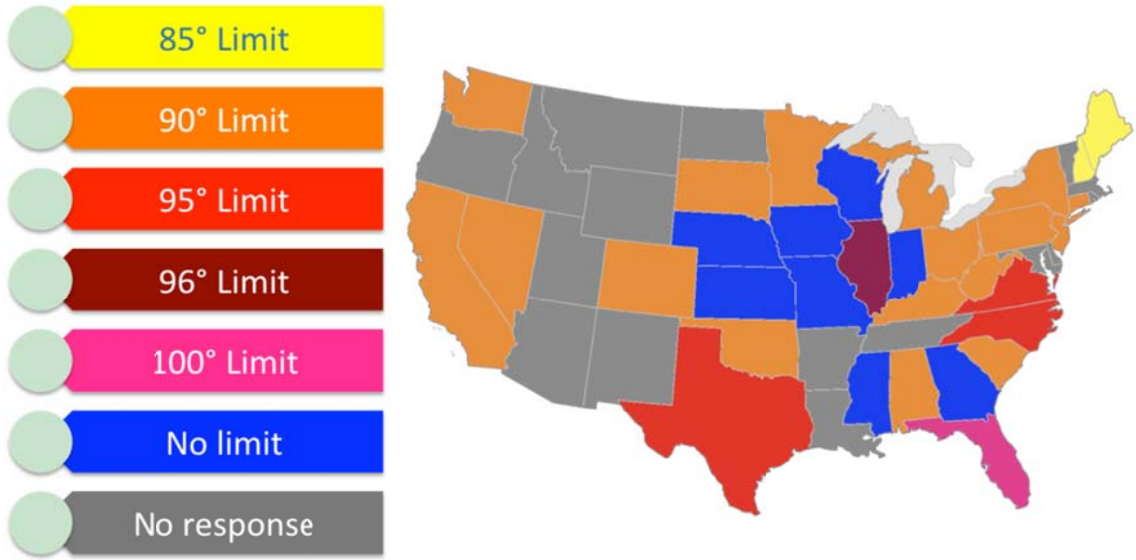
Figure 5. UIUC survey results for maximum placement temperature ($^{\circ}$F).
Note $^{\circ}C = (^{\circ}F - 32) \times 5/9$.
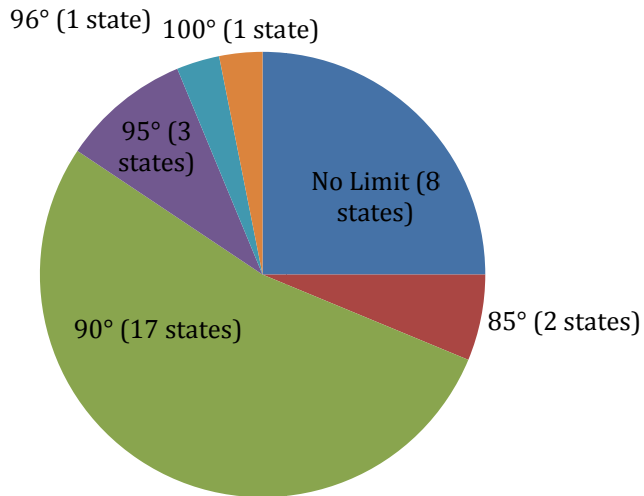
Figure 6. UIUC survey results for maximum placement temperature ($^{\circ}$F).
Note $^{\circ}C = (^{\circ}F - 32) \times 5/9$.

3.4.3 Hot Weather Concrete Distress Results

Of the 32 states that responded, 18 reported no damage/distress, and 14 reported some kind of distress that likely is a result of high concrete mixture temperatures. The states that reported pavement damage had different types of distresses. The states with deterioration were divided into categories based on their description of the problems: shrinkage cracking only, possible structural cracking, surface deterioration such as the pop

outs and crazing like IDOT has noted. Two of the states' descriptions were too vague to categorize. As seen in Figure 7, only six states reported shrinkage cracking, four states stated structural cracking is possibly due to the hot weather concreting, and two states had surface deterioration such as poor finishing, pop outs, or crazing.
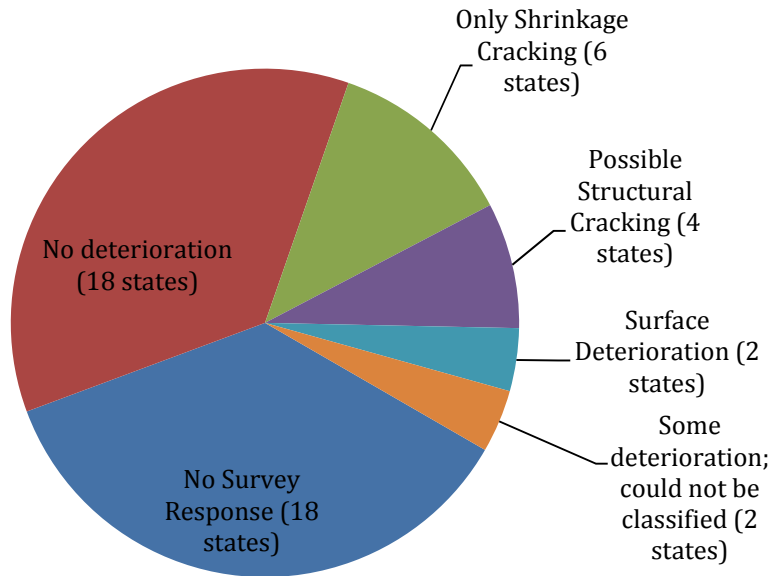
Figure 7. Results from survey of concrete pavement deterioration from hot weather concreting.

3.4.4 Survey Trends

Based on the survey results for concrete placement temperature and presence of distresses, some rough trends can be observed. It should be stressed that, due to the relatively small survey sample size, these trends are by no means conclusive. However, they do suggest some interesting conclusions regarding hot weather policies and concrete pavement distress.

There seems to be some relationship between those states that specify a 90°F (32.2°C) or lower maximum allowable temperature limit and those states that report no pavement distress. Only five of the 18 states which have a 90°F (32.2°C) or 85°F (30°C) limit report any damage. This is illustrated in Figure 8. The survey results suggest that the states that have a maximum allowable limit of 90°F (32.2°C) or lower are controlling the hot weather concreting problem fairly well.
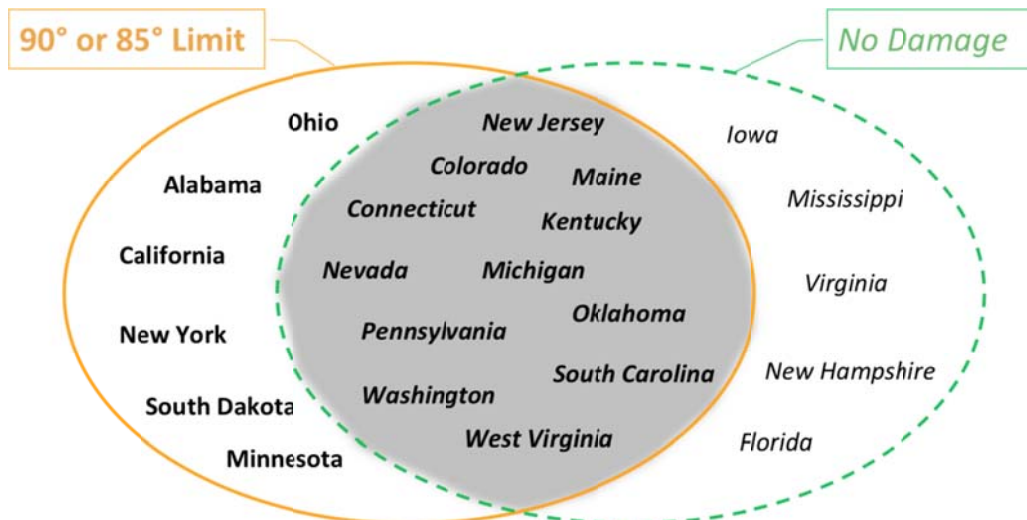
Figure 8. Venn diagram representing the apparent trend between states that have a 90°F (32.2°C) or lower limit and report no deterioration. The dark shaded area indicates states that report both a 90°F (32.2°C) or lower limit and no deterioration.
Note °C = (°F – 32) x 5/9.

There is also some relationship between those states with no limit and the states that reported pavement distresses. Only two of the eight states with no temperature limit also report no distress as illustrated in Figure 9. From the survey results, it would seem that having no limit on concrete temperature may lead to increased likelihood of distressed concrete.
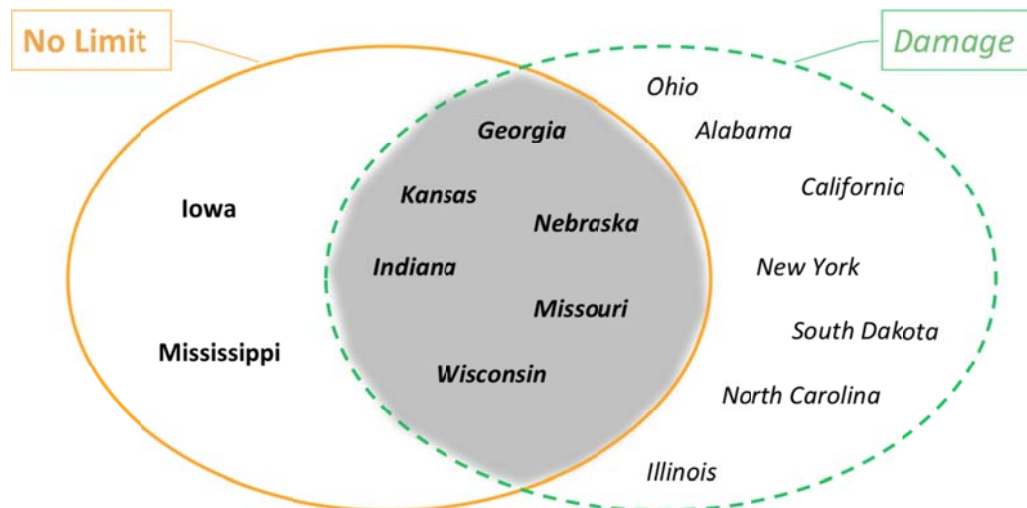
Figure 9. Venn diagram representing the trend for states with no limit and pavement damage. The dark shaded area indicates states that report both no temperature limit and deterioration.

3.4.5 Selected Survey Results – States with Interesting Policies

Other than temperature limits, many states have other practices designed to control hot weather concreting. Florida's policy of verifying laboratory tests for concrete to be used in hot weather has already been described. Many states (including Illinois) require retarder in all their concrete during hot weather. Illinois requires retarders when plastic temperatures exceed 85°F, but, at least in Illinois, this policy does not appear to be widely enforced. This was determined by the responses of IDOT employees who stated that the enforcement of the retarder requirement varies by district.

A few states, such as New Jersey and North Carolina, specify maximum plastic temperatures as well as maximum ambient air temperatures. Most states have stricter requirements for structural concrete than pavement concrete. Mississippi (which has no maximum temperature for pavement) restricts their structural concrete to 90°F (32.2°C), unless it contains more than 20% fly ash, in which case it is allowed to rise up to 95°F (35°C). Minnesota's response indicated that though their limit is 90°F (32.2°C), they generally allow temperatures up to 92°F (33.3°C) as an unwritten rule.

Five states mentioned the use of the evaporation rate nomograph in the American Concrete Institute's "Specification for Hot Weather Concreting" (ACI 305.1-06). However, only two states (Maine and Nebraska) use it for all pavements while the other three (New Jersey, Indiana, and Kansas) use it for bridge decks only. Indiana requires the use of an evaporative retarder on the concrete surface if the evaporation rate is too high. In the survey response from Texas, it was noted that many projects specify the use of class F fly ash between April and October. Maine requires decreased haul times depending on the mixture plastic temperature.

3.4.6 Cooling Practices

When analyzing the survey responses regarding cooling practices, it is important to note that these are merely the common practices in the state, and most states allow the contractors to use any cooling method they choose as long as the specifications are obeyed.

Of the 32 responses, 21 states report using ice in their concrete and 18 states report cooling constituents of the concrete (water, aggregate, etc.). Other practices include night or morning placement (seven states), use of liquid nitrogen (five states), as well as windbreaks, fogging, or wetting trucks, sub-grade, or reinforcing steel. Five of the states that responded said that they have no standard practices or do not cool their concrete.

3.4.7 Follow-up Questions Neighbor States with No Limit

After the initial analysis of the survey responses, it was noticed that several of Illinois' neighbors (with similar climatic conditions) have no temperature limit on their concrete. These states are Wisconsin, Missouri, Iowa, and Indiana. A short follow-up survey was sent to those states. The follow-up questions explained that although Illinois has a 90°F (32.2°C). limit for delivered plastic concrete, it has experienced problems related to loss of slump, poor finishing, and addition of excess water. The follow-up questions were designed to determine how the neighboring states deal with concrete placement in hot weather, specifically if they have any other types of special hot weather requirements, such as decreased haul times, workability requirements, or strict contractor penalties for inferior work. Additionally, the neighboring states were asked for examples of either very good or very bad hot weather concreting experiences. The answers to the follow-up questions are summarized here.

Wisconsin's response was that their specifications primarily focus on end results of the in-place pavement product, placing the risk and responsibility for controlling the production process on the contractor. They recognize that high temperatures in concrete are undesirable and increase the risk for problems. Workability and set problems are enforced by disincentive pay adjustments for poor ride quality. Adding extra water to compensate for workability or set problems is similarly penalized by a resulting low compressive strength measurement. Air content problems result in substantial penalties or pavement removal. Random cracking resulting from concrete thermal or drying shrinkage or improper saw-cut timing requires full-depth repairs of the pavement at the contractor's expense. Wisconsin encourages their contractors to use HIPERPAV software to monitor and manage stress levels in the pavement. They report most problems when the concrete is placed during hot weather, immediately followed by a strong cold front or thunderstorm. This thermal shock condition causes the pavement to crack.

Missouri has no requirement for maximum concrete temperature, and also has no restriction on discharge times of concrete. Rather, they specify that concrete, as delivered, must be such that "the handling and discharge of concrete shall not cause segregation or damage to the concrete and will allow placement with a minimum of handling. All handling and discharge shall occur prior to initial set of the concrete. Missouri also specifies a maximum of 300 revolutions of the truck after mixing. Missouri does not specify workability requirements for the concrete, and only focuses on the end product to make acceptance decisions. This places the responsibility on the contractor, who must ensure that the proper strength, thickness, air content, and smoothness are obtained. Since these results determine the contractor's pay, Missouri has been able to obtain consistently good concrete while allowing a great deal of flexibility for the contractor. An example of poor concrete is a project in August in the St. Louis area. The contractor was having trouble obtaining a stable air void structure during hot weather. Several different brands of air entrainer were tried with no success, and the contractor was about to switch to night work when the heat wave ended and the problem resolved itself.

Iowa responded that they had problems in the mid 1980s with Type I cements, 15% class C fly ash, and certain water reducers during hot weather. Since the mid 1990s, the majority of their paving has been done with blended cements (IS or IP) combined with 15-20% class C fly ashes. They haven't had hot weather concrete problems since they started using this combination. Their contractors may also use a retarder with these combinations. As far as the Iowa respondent knew, there could be problems with this combination, but not nearly as many problems as before they switched. He gave an example of a project using class IS and 10% class C fly ash in 1997 when the "belt placer broke down and the concrete laid on the grade had sat there for over the ½ hour limit. As the pavement came over that concrete, it still came out behind the paver without any bugholes." Iowa appears to be able to control their concrete in hot weather by using significant quantities of mineral admixtures.

A response from the Indiana DOT was not obtained.

3.4.8 Survey Conclusions
The following conclusions are drawn from the results obtained in the survey:

- Most states specify a maximum allowable plastic concrete temperature of 90°F or lower. However, the time at which this temperature must be taken, for example at delivery or before final placement, is not always clearly specified.
- Illinois is the only state that specifies two separate maximum plastic concrete temperatures, one at delivery before placement at 90°F (32.2°C) and one as placed 96°F (35.5°C).

- Several states, including nearly all of the neighboring states to Illinois, do not specify a specific maximum allowable temperature for plastic concrete. Most often, these states specify a quality end product regardless of temperature. This approach places burden on the contractor, but also allows for flexibility for the contractor to achieve the quality end product by whatever means he/she chooses.
- With regard to concrete damage thought to be caused by high mixture temperatures, most states that responded to this question report no known damage. Of the states that do report damage, most is in the form of shrinkage or structural cracking.
- Although it is not appropriate to draw definitive conclusions from a limited survey sample, the survey results do suggest that the states that have a maximum allowable limit of 90°F (32.2°C) or lower are controlling the hot weather concreting problem fairly well, as these states also report low occurrence of temperature related distress.
- Many states specify other practices, such as required use of retarder, fly ash, haul time limits, etc., in addition to maximum allowable plastic concrete temperatures.

CHAPTER 4 ILLINOIS SITE VISITS

The research team has surveyed the experiences of IDOT field personnel about current practice and experience with concrete cast at higher temperatures. The research team conducted three visits to roadway sites with suspected hot weather concrete problems. This task required close assistance from the IDOT Bureau of Materials and Physical Research (BMPR) and the district engineers.

The three site locations visited were in Chicago, Peoria (three projects within Peoria were seen on the same day), and Collinsville, Illinois. These projects include major interstate highway pavement and a barrier wall, interstate exit ramps, local roads, and a driveway. Each section is presented separately below, with as many details of pour conditions and apparent damage to date.

4.1 DAN RYAN EXPRESSWAY, CHICAGO, IL

This section, along the Dan Ryan Expressway (Interstate 90/94) response express lanes in Chicago, was visited on January 8, 2008, by Drs. John Popovics and Jeff Roesler from UIUC, hosted by IDOT representative Mel Kirchler.

4.1.1 Placement Conditions

The CRCP section which IDOT reported as having irregular cracking patterns and surface characteristics was cast in the summer of 2007. According to IDOT engineers, the ambient temperatures during placement were not very warm (maximum temperature 73°F (22.7°C) under cloudy to partly cloudy skies), yet several delivered concrete loads were rejected because the delivered fresh concrete temperatures were above 94°F (34.4°C). Higher than normal slump loss, and rapid concrete setting for those mixtures were noted by field personnel. The field personal attributed the high mixture temperatures to reports of hot cement clinker at the batch plant (St. Mary's) that supplied the cement. A similar mixture design and casting procedure was used for this section as in other locations and the previous year. Curing of the concrete was done with a white membrane curing compound applied to the pavement surface after initial set. The concrete mixture had a total cementitious materials content of 535 lbs/cy, which includes 20% slag replacement. The mix design had a water to cementitious materials ratio of 0.42. The compressive strength samples of this mixture showed normal, acceptable values at seven days. The following observations were made during the site visit and discussions with the District 1 materials engineer, Mel Kirchler.

4.1.2 Deterioration

IDOT reported a larger amount of transverse cracking within one day after casting. Examples of the transverse cracking are shown in Figures 1 and 2. This section of pavement is CRCP, so some transverse cracking is expected due to the continuous longitudinal steel in the pavement. The crack spacing was reported to be much closer than normal (6 inches instead of 2-4 feet), although it was difficult to make any measurements due to the high volume of traffic. Because the weather conditions at the time of casting were not excessive (neither very warm nor very sunny), the increased cracking is attributed to the high mixture temperatures. The surface appearance of the concrete was different than adjacent sections which could be attributed to higher placement temperatures and the subsequent difficulty in closing the surface of the fresh concrete.

Figure 10. Closely spaced transverse cracks on Dan Ryan Expressway.

Figure 11. Close up view of transverse crack on Dan Ryan Expressway.

4.2 I-74 RAMP, PEORIA, IL

This section, an off-ramp from interstate 74, was visited on March 6, 2008, by Dr. John Popovics, Dr. Jeff Roesler, and Carrie Peterson from UIUC, as well as IDOT representatives Derek Parish and Stephen Worsfold.

4.2.1 Conditions

The pour on the ramp started at about noon on June 25, 2005. According to IDOT engineers, the concrete temperatures were over 93°F (33.8°C). The concrete experienced severe slump loss in transit because it was delivered in open tandem trucks, and traffic difficulties on site required the trucks to wait 45-60 minutes before they could dump their load. Four of the trucks on the project had to be rejected because of excess time since batching. Because of the severe slump loss, the concrete was difficult to finish and the contractor was not able to put a suitable surface texture on it. However, once traffic issues were resolved and fresher loads of concrete were placed, the pour went better.

4.2.2 Deterioration

As previously mentioned, the contractor was unable to put the proper texture into the concrete, so the surface is very smooth which is undesirable for proper skid resistance. This is especially problematic for a highway exit ramp where traffic needs to decelerate rapidly. In addition to the lack of proper tining, the concrete is starting to show some fine crazing. Though this is not a problem yet, it could worsen into surface scaling. This crazing is most likely due to the overworking of the surface by the finishers. This could have included addition of water to the surface to aid in finishing which ultimately weakens the surface concrete. The surface of the exit ramp concrete is shown below in Figure 12.

Figure 12. Crazing and poor tining on exit ramp jointed concrete pavement.

4.3 DRIVEWAY PAVEMENT OFF US-150, PEORIA, IL

This pavement was a small piece of driveway pavement near the IDOT district office that was an example of poor summer paving technique. UIUC representatives Dr. John Popovics, Dr. Jeff Roesler, and Carrie Peterson, accompanied by IDOT representatives Derek Parish and Stephen Worsfold, visited the pavement on March 6, 2008.

4.3.1 Placement Conditions

This driveway was poured in August 2002, starting at about 4:00 in the afternoon. The assistant resident engineer (RE) believed that the load of concrete was one previously rejected from a nearby pour because of excess time since batching, about 90 minutes. After the truck was rejected, it returned within 45 minutes, which the RE believes was not enough time to come back with a fresh load. The concrete was placed at the driveway site anyway, with very high slump, as if the truck driver had just added water to push the slump into an acceptable range. As soon as the concrete was struck off, it began setting and the finishers had a very difficult time finishing and brooming the surface.

4.3.2 Deterioration

These concrete slabs have almost no surface texture and significant popout distress across the entire surface. A petrographic examination was not done, but this could be the result of very poor air void structure at the surface, leading to freezing damage during the winter. This damage can be seen in Figure 13.

Figure 13. No surface texture and significant pop-out distress.

4.4 CRUGER ROAD, WASHINGTON, IL

This local road was poured in two stages, such that the eastbound side was poured in slightly cooler weather in the fall, and the westbound side was poured in hot summer conditions. Since the concrete mixture and pavement design were presumably the same as well as the loading, this section makes an ideal side-by-side comparison of the effects of higher concrete placement temperatures. It was examined by IDOT representatives Derek Parish and Stephen Worsfold and UIUC representatives Dr. John Popovics, Dr. Jeff Roesler, and Carrie Peterson on March 6, 2008.

4.4.1 Placement Conditions

Westbound Cruger Road was poured on August 29 and 30, 2007. The ambient air temperature ranged from 72 to 89°F, and the weather was partly cloudy on the first day, and partly cloudy then sunny on the second day. The concrete was transported in agitating trucks, and the average haul time reported was 25-30 minutes. Concrete temperatures reported on the first day were between 86 and 94°F (30 and 34.4°C) and 84 and 92°F (28.8 and 33.3°C) on the second day. Concrete slump varied from 1.25 to 2.75 inches, and air content ranged from 3.5% to 7.8%.

Eastbound Cruger Road was poured on September 20 and 21, 2007. Weather conditions were clear on both days, with ambient temperatures between 66 and 90°F (18.8 to 32.2°C). Concrete temperatures between 83 and 91°F (28.3 and 32.7°C) were reported. Concrete was still transported in agitating trucks, and the average haul time was reported as approximately 25 minutes. Concrete slump varied from 1.25 to 2.00 inches, and air content ranged from 3.6% to 8.7%. Other than the ambient weather condition differences, each direction of the road was very similar in terms of materials and structural design.

4.4.2 Deterioration

Even though the concrete temperatures were lower during the pour of eastbound Cruger road, it shows more deterioration than the westbound lane. The eastbound lane shows overworking of the surface, crazing, poor tining and surface texture, and very poor contraction joint quality. Whether related to warm weather or not, the joints were sawed too early, and the edge of the joint was badly spalled. A poorly sawn joint and the poorly broomed texture can be seen in   Figure 14. Figure 15 shows a poor finished surface, and Figure 16 shows an adequately finished surface for Cruger Road.

Figure 14. Excessive joint spalling on the eastbound direction of Cruger Road, as well as poor surface texture.

Figure 15. Eastbound Cruger Road, showing areas of poor surface friction, which are reflecting the sunlight.

Figure 16. Westbound Cruger Road with proper surface texture (left lanes). The discoloration in the westbound lane is merely a cosmetic flaw.

4.5 I-64 BARRIER WALL, FAIRVIEW HEIGHTS, IL

A barrier wall dividing eastbound and westbound traffic on Interstate 64 in Fairview Heights, Illinois (east of St. Louis) was examined by UIUC and IDOT personnel on July 16, 2008. IDOT representatives included Derek Parish, Richard Mauch, and several IDOT workers who were present during the original project. The UIUC representatives were Dr. Jeff Roesler, Dr. John Popovics, and Carrie Peterson. The visit to the roadway itself was followed by a brief stop at the local IDOT district office in Collinsville, where several small cores from some of the repaired cracks at the site were obtained.

4.5.1 Placement Conditions

Concrete placement at the same barrier wall occurred over a period of time during several seasons, but the most serious problems occurred during summer months with hot weather. The delivered concrete temperature, as reported, was generally below 90°F (32.2°C) and always below 96°F (35.6°C); however, the IDOT engineer on site remembers seeing concrete temperatures in the upper 90s. The slump and air content were reported as within acceptable limits. Open tandem (dump trucks) were used to transport the concrete, and haul times were relatively short and within the allowable limits.

The mix design used was not a standard IDOT mix, since the contractor was from Missouri and had a mixture that he thought would work well. Some of the mixtures used entirely portland cement, and some of them incorporated fly ash. It was not clear when these were used, but the concrete performance seems to be the same for the fly ash mixes and the straight cement mixtures. The concrete had a water to cementitious materials ratio of 0.35, and a retarder was used during the summer months. The contractor also sprinkled the aggregate stockpiles during the summer to cool it, and also employed ice in the mix at times.

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4.5.2 Deterioration

The major problem with the barrier wall on I-64 was the frequent occurrence of transverse cracking in the barrier wall that appeared very quickly after concrete placement, sometimes less than one hour after the slip-forming paver placed the concrete (see Figure 17, Figure 18, and Figure 19). The contractor attempted to saw contraction joints earlier to relieve the developing tensile stresses, but the transverse cracks appeared before the concrete was mature enough to saw. Finishers attempted to close the cracks, but because the cracks were wide and deep, this was not successful. After the concrete casting, the contractor had to come back and repair many of the severe cracks with epoxy injection.

Areas of cracking also showed poor consolidation, possibly due to the irregular barrier wall geometry used with a slip-forming process. Because both the concrete and the air around it were hot, and the cracks formed before the weather cooled, it is unlikely that these transverse cracks were thermal cracks. Thermal cracks develop as the concrete cools rapidly which did not appear to be the case. The IDOT field personnel noted that excessive moisture evaporation was likely the reason for the transverse cracking. The concrete mixture already had a very low water to cementitious materials ratio and was transported in open tandem trucks. The barrier wall project was continued through several seasons, and the transverse cracking only appeared at the very beginning of the project (where the rebar cage had to be redesigned because of significant consolidation problems) and during the summer months when the weather was hot and sunny. The area was also continuously windy because of highway traffic adjacent to the construction project. Barrier walls are also more susceptible to evaporation problems because of their high surface area to volume ratio.

Figure 17. A crack in the barrier wall that appears to have been covered with a thin layer of paste, but has cracked through again.

Figure 18. Cracking in the barrier wall could be quite frequent, occurred regardless of contraction joints, and initiated at the top of the barrier wall.

Figure 19. A severe crack in the barrier wall. Poor consolidation can be seen in the cracked off portion, and the contraction joint is not cracked through.

4.6 SITE VISIT SUMMARY

The concrete at the three sites visited exhibits some combination of excessive cracking, rapid slump loss, early setting time, and poor finishability. The common feature in these three sites is that concrete temperatures were high, either very close to or above 90°F (32.2°C). Most of the sites visited were slip-form paving projects, and therefore had a low water to cementitious materials ratio. It should be investigated further if this is an indicator of increased susceptibility to hot weather concreting problems. The following questions should be addressed in order to gain more insight into this problem:

- Is there another important common theme between these sites and their problems?
- Would enforcement of the 90°F (32.2°C). delivered limit have prevented the cracking?
- Would other types of quality control have prevented the damage?
- What would the most efficient and cost-effective solution have been?

CHAPTER 5 EXPERIMENTAL TESTS

Experimental tests were carried out in order to better understand the effects of high placement temperature on concrete. The tests are divided into two different series: tests on field samples (cores) and tests on laboratory samples. Core samples were obtained from several different locations at one of the site visit locations (Peoria). The primary goal of the tests was to determine what differences (if any) were present between concrete poured in hot weather conditions and those poured under normal temperatures. In the laboratory test series, fresh concrete mixtures were subjected to the high temperature mixing procedure developed by Florida DOT. Air content and slump were measured before and after the 90-minute mixing procedure. Compressive and splitting tensile strengths of cast samples were measured. The principal goal of the tests was to determine the effects of mineral and chemical admixtures on the performance of fresh concrete at high temperatures.

5.1 TESTS ON FIELD SAMPLES

The samples obtained from the field were analyzed by standard resonance frequency analysis and by the standard test for density, absorption, and voids, i.e. the "boil" test. The resonance frequency test (ASTM C 215) determines an overall dynamic Young's modulus of elasticity of the concrete sample; Young's modulus is affected by distributed cracking (damage) levels within the material, among other parameters. The boil test (ASTM C 642) determines the bulk density, absorption, and the volume percentage of permeable pore space within the concrete sample. Changes in permeable void space within the material serves to indicate changes in damage levels and pore structure (i.e. due to compaction) within the material.

5.1.1 Roadway Test Sample Locations

In an initial study, the research group from the University of Illinois conducted three visits to roadway sites with suspected hot weather concrete problems. These projects included major interstate highway pavement and a barrier wall, interstate exit ramps, local roads, and a driveway. Core samples of these projects were obtained for the purpose of performing a series of laboratory tests to identify any possible problems or differences between concretes poured at lower temperatures. The primary goal of the tests was to determine what differences (if any) were present between the top section of the cores, which theoretically should lose the most moisture and undergo significant finishing operations, and the middle sections of the core sample. The three sites are described here.

Cruger Road

Cruger Road is a local two-lane road in Washington, IL. The opposing lanes were poured approximately one month apart in August and September, 2007. Concrete placed on the westbound lane at the core location had a plastic temperature of 94°F (34.4°C) and measured air content of 2.8%. Concrete placed on the eastbound lane at the core location had a plastic temperature of 91°F (32.7°C) and air content of 7.2%. Although both slabs were poured with high concrete plastic temperature, only the eastbound lane of the road visually demonstrates negative effects as a result of the temperature: poor surface finish, erratic tining depth, and joint spalls, as well as some crazing.

US Route 150

The cores taken from US Route 150 are from two different projects: a driveway pour which was placed in hot weather, and the concrete gutter nearby. There is no testing information available on the driveway pavement, except the inspector's belief that it was

placed at least 135 minutes after the addition of water. The nearby gutter pavement was placed about a month later and had a temperature of 67°F (19.4°C) when it was placed, but it has a different aggregate type from the driveway pavement and it is most likely not the same concrete mixture.

Interstate 74

Cores were taken from a ramp off I-74 which was placed in hot weather as well as a nearby shoulder which was a similar mixture design and the same transportation and placement technique. The two sections were placed approximately four months apart. The core from the ramp section had plastic temperatures of at least 93°F (33.8°C), and the shoulder section had plastic temperatures of about 60°F (15.5°C).

5.1.2 Core Sample Summary

The cores and basic information are summarized in Table 2. After they were received, the rough bottom surfaces were trimmed, and they were stored at room temperature and allowed to dry in the ambient indoor air.

Table 2. Summary of IDOT District 4 core samples. Note°C = (°F – 32) x 5/9 and 1 inch = 25.4 mm

Road/Location	Pour	Distress	Plastic Concrete Temperature	Core label and approximate as-received lengths
Cruger Road	WB	None	94°F	CRWB1: 8.01" CRWB2: 7.94"
	EB	Poor finishing, crazing	91°F	CREB1: 7.96" CREB2: 7.79"
US150	Driveway	Poor finishing, popouts	Unknown**	150D1: 5.49" 150D2: 5.89"
	Ramp/Gutter	None*	67°F	150R1: 8.10" 150R2: 8.18"
I-74	Ramp	Poor finishing, crazing	93+°F	74R1: 8.34" 74R2: 3.37" 74R3: 5.21"
	Shoulder	None*	60°	74S1: 3.99" 74S2: 4.66"
*Sites were not surveyed by UIUC personnel, but were provided by IDOT as samples of nearby, similar "good" concrete, and are therefore assumed to not be showing any distresses. **Concrete is suspected to be rejected from a nearby site. The IDOT engineer suspects it was placed at least 135 minutes after addition of water.				

5.1.3 Resonance Frequency Analysis

The resonance frequency method is a non-destructive test to determine the dynamic elastic modulus (E) of concrete cores or prisms. It is sensitive to changes in damage or micro-cracking content, and is frequently applied to evaluate freezing-thawing damage in concrete. However, E is also affected by the type of aggregate and moisture content within the concrete.

Testing Procedure

Resonance testing was performed on the core samples indicated in Table 2 after the rough ends were trimmed with a wet saw. Then the cores were cut to a length of 10 cm from the finished surface, and resonance testing was performed again. Finally, the trimmed cores were cut in half into two 5 cm lengths and the resonance testing was performed a final time. The test for density, absorption, and voids was then performed on the 5cm long samples; a description of this test is presented in a later section.

The resonant frequency testing procedure is outlined in ASTM C 215, and requires an impactor, and accelerometer and a digital oscilloscope or frequency counter. The core sample can be excited in vibration with the longitudinal, transverse, or torsional vibration modes. The longitudinal excitation mode was used for this test. According to the testing standard, a 2:1 aspect ratio of the sample is required in order to compute E from the test

results. However, due to the size of the field samples, a 2:1 aspect ratio was not achieved. However, E values obtained from samples with roughly equivalent aspect ratios can be used to establish relative differences in material stiffness. Since the goal of this effort is to show differences in E among the different samples, rather the absolute value of E, data from the non-standard sample aspect ratios is useable as long as the aspect ratios are kept fairly constant.

Results

Table 3 shows the dynamic Young's moduli obtained with the resonance test for 10cm core samples. A statistical analysis of the results follows. The results from the 10cm core samples show that all of the samples show roughly equivalent aspect ratio, ranging between 1:0.85 to 1:1.1. This range of aspect ratio ensures that E values obtained from the resonance tests are comparable to each other. The samples are comprised of either limestone or river rock (gravel) coarse aggregate, which is visually determined from the exposed core surface.

The data were analyzed using a statistical T test, which is appropriate for a limited number of samples (< 30) drawn from a population that is described by the normal ("bell curve") distribution. We can estimate if two sample populations are distinct from each other, to a certain level of statistical confidence, using the T test knowing the sample mean and variance (square of standard deviation) of our test samples, for a specific number of samples (degrees of freedom) [15]. For a two-tailed test, the degree of statistical confidence in percent is determined by $100(1-2*\alpha)$, where α is the area under the T distribution curve above a computed T value. The T value is computed as follows:

$$T = (x_1 - x_2)/(s_p (1/n_1 + 1/n_2)^{0.5})$$

where x_1 is the sample mean of sample 1, x_2 is the mean of sample 2, s_p is the combined standard error of estimate for both samples, n_1 is the number of units in sample 1 and n_2 is the number of units in sample 2.

Table 3. Dynamic Modulus results for 10cm core lengths.

Core Label	Hot / Cold	Short Label	Mass (g)	Diameter (cm)	Average Length (cm)	Aggregate Type	Modulus (GPa)*
US 150 Driveway #1	Hot	150D1	1824.74	10.03	10.48	Limestone	31.57
US 150 Driveway #2	Hot	150D2	1762.38	10.03	10.16	Limestone	27.99
US 150 Ramp Gutter #1	Cold	150R1	2030.45	10.03	11.03	River Rock	36.00
US 150 Ramp Gutter #2	Cold	150R2	2009.50	10.03	10.91	River Rock	35.25
I-74 Ramp A-1 #1	Hot	74R1	1805.86	10.03	10.16	Limestone	31.13
I-74 Ramp A-1 #2	Hot	74R2	1516.72	10.03	8.57	Limestone	31.06
I-74 Ramp A-1 #3	Hot	74R3	1788.27	10.03	10.16	Limestone	32.49
I-74 Shoulder #1	Cold	74S1	1733.31	10.03	10.08	Limestone	30.44
I-74 Shoulder #2	Cold	74S2	1817.35	10.03	10.45	Limestone	33.09
Cruger Road EB damage #1	Hot	CREB1	1914.12	10.03	10.48	River Rock	36.80
Cruger Road EB damage #2	Hot	CREB2	1902.57	10.03	10.36	River Rock	39.96
Cruger Road WB no damage #1	Hot	CRWB1	1947.46	10.03	10.64	River Rock	39.98
Cruger Road WB no damage #2	Hot	CRWB2	1934.79	10.03	10.48	River Rock	42.12

* 1 GPa = 145000 psi

Limit values of T are established for a given α value and number of samples, which are obtained from reference books. In this study, a 95% confidence level is assumed, meaning that α = 0.025. If the computed T is greater than the limit value of T for α = 0.025 and the number of samples, then the hypothesis – that the means of samples 1 and 2 are distinct with 95% confidence level – is accepted. If the computed T is less than the limit value of T, then the hypothesis is rejected.

Since the aggregate type is known to influence the E values obtained from vibration resonance, a statistical analysis of this influence was first carried out. The statistical data from the seven limestone aggregate concrete samples and six river rock aggregate concrete samples are presented in Table 4. A first look at the data in Table 4 suggest that the river rock samples have significantly higher E values, regardless of the plastic concrete temperature condition of the sample. The T distribution analysis confirms, with over a 95% confidence level, that the samples with different aggregate types have distinct average E value. This means that E values are significantly influenced by aggregate type. Next the influence of concrete plastic temperature was analyzed; those data are also shown in Table 4. Note that the samples from the apparently undamaged sections of Cruger Road are considered to be "cold" for the purpose of this analysis, even though technically they did exhibit high plastic temperatures. A first look at the data in Table 4 suggests that the "cold" samples have higher E values, regardless of the aggregate type. However, this inference is

not supported at the 95% confidence level according to the T test results. This inference is supported at the 80% confidence level though.

Table 4. Statistical results of dynamic modulus values obtained from resonance of 10 cm core samples, with regard to aggregate type and temperature conditions.

Groups	Count	Mean (GPa)*	Variance(GPa^2)
All, limestone	7	31.1	2.70
All, river rock	6	38.4	7.40
All, hot	7	33.0	16.3
All, cold	6	36.2	18.6

* 1 GPa = 145000 Psi

Since E values are significantly affected by aggregate type, the influence of distributed damage owing to high plastic concrete temperatures on the obtained E values may be masked. Thus the data from comparable (with regard to aggregate) samples was analyzed statistically, and the results are shown in Table 5. The T test analysis on these data shows that there is no meaningful statistical difference in dynamic E between hot and cold samples within each aggregate type. Note that the sample sizes here are small, so it is more difficult to draw strong statistical conclusions. Nevertheless, based on the provided concrete samples, the data show that overall dynamic E is not affected meaningfully by the plastic concrete temperature conditions. Further we conclude that the distributed microscopic damage state within those samples is not significantly different.

Table 5. Statistical results of dynamic modulus obtained from resonance of 10cm core samples with regard to aggregate type and temperature condition.

Groups	Count	Mean (GPa)	Variance(GPa^2)
Limestone, hot	5	30.85	2.878
Limestone, cold	2	31.77	3.511
River Rock, hot	2	38.38	4.993
River Rock, cold	4	38.34	10.67

* 1 GPa = 145000 Psi

Table 6 shows the results for the 5cm core samples. The data from the halved core samples were statistically analyzed, using the T test, in order to determine if there is a difference in E values between the upper section of the core sample (near the pavement surface) and the mid-section of the same core sample for different plastic concrete temperatures. The statistical data are shown in Table 6. Although relatively small sample

population size and relatively high variance do limit the statistical analysis, some statistical inferences can be made using the T test as a basis. The data suggest that the top sections of the core samples have significantly higher E values, regardless of the plastic concrete temperature condition or aggregate type of the sample. The T test analysis confirms this, with over a 95% confidence level: top sections of the samples show higher E, regardless of other attributes. This outcome of the test was not expected since high temperatures at the surface would create more near surface problems than towards the middle of the concrete slab. The higher dynamic modulus could be due to uneven aggregate distribution throughout the core sample depth (more aggregate at the top), less air voids near the surface, or enhanced concrete compaction near the pavement surface where vibrators are located. This statistical distinction at the 95% confidence level between top section and mid-section E values is also seen when only the "cold" samples are analyzed. All other sample sets, when comparing aggregate types and temperature conditions, show too much variability to draw any meaningful statistical conclusion about the differences between top and midsection sections of the same core. Figure 1 shows these findings, where the E values from the top section of each core are plotted against those of the mid section for all the samples. Most of the data points lie to the right of the line of equality (LOE), which confirms the conclusion that the top sections show higher modulus, regardless of other parameters, to a high degree of statistical confidence. However, no clear distinction between hot and cold plastic temperatures is seen. As both hot and cold samples show about the same level of scatter about the line of equality.

Table 6. Dynamic Modulus Results for 5cm Core Lengths.

Core Label	Hot / Cold	Short Label	Mass (g)	Diameter (cm)	Average Length (cm)	Aggregate Type	Modulus (GPa)**
US150 Driveway #1	Hot	150D1-TOP	891.62	10.03	5.16	Limestone	27.60
US150 Driveway #1	Hot	150D1-MID	885.61	10.03	5.04	Limestone	28.32
US150 Driveway #2	Hot	150D2-TOP	841.20	10.03	4.80	Limestone	25.63
US150 Driveway #2	Hot	150D2-MID	875.50	10.03	5.00	Limestone	28.55
US 150 Ramp #1	Cold	150R1-TOP	1042.71	10.03	5.68	River rock	39.64
US 150 Ramp #1	Cold	150R1-MID	937.46	10.03	5.08	River rock	28.98
US 150 Ramp #2	Cold	150R2-TOP	1077.69	10.03	5.91	River rock	38.78
US 150 Ramp #2	Cold	150R2-MID	881.15	10.03	4.76	River rock	29.26
I-74 Ramp A-1 #1	Hot	74R1-TOP	967.70	10.03	5.48	Limestone	36.95
I-74 Ramp A-1 #1	Hot	74R1-MID	789.67	10.03	4.56	Limestone	25.79
I-74 Ramp A-1 #2	Hot	74R2-TOP	940.02	10.03	5.32	Limestone	38.11
I-74 Ramp A-1 #2*	Hot	n/a	no data	no data	no data	no data	no data
I-74 Ramp A-1 #3	Hot	74R3-TOP	874.42	10.03	4.92	Limestone	31.20
I-74 Ramp A-1 #3	Hot	74R3-MID	864.16	10.03	4.92	Limestone	30.04
I-74 Shoulder #1	Cold	74S1-TOP	884.22	10.03	5.16	Limestone	29.75
I-74 Shoulder #1	Cold	74S1-MID	800.33	10.03	4.72	Limestone	26.70
I-74 Shoulder #2	Cold	74S2-TOP	914.81	10.03	5.23	Limestone	30.35
I-74 Shoulder #2	Cold	74S2-MID	854.11	10.03	4.92	Limestone	30.08
Cruger Road EB #1	Hot	CREB1-TOP	999.05	10.03	5.48	River rock	41.22
Cruger Road EB #1	Hot	CREB1-MID	866.01	10.03	4.80	River rock	34.58
Cruger Road EB #2	Hot	CREB2-TOP	977.00	10.03	5.40	River rock	33.46
Cruger Road EB #2	Hot	CREB2-MID	873.82	10.03	4.76	River rock	25.68
Cruger Road WB#1	Hot	CRWB1-TOP	958.18	10.03	5.16	River rock	33.12
Cruger Road WB#1	Hot	CRWB1-MID	938.33	10.03	5.24	River rock	32.05
Cruger Road WB#2	Hot	CRWB2-TOP	997.91	10.03	5.36	River rock	35.34
Cruger Road WB#2	Hot	CRWB2-MID	884.10	10.03	4.84	River rock	34.74
	*Core length was too small for two 5 cm (1.97 inches) sections ** 1 GPa = 145000 Psi						

Table 7. Statistical Results of Dynamic Modulus Obtained from Resonance of Sawn 5cm Core Samples with Regard to Aggregate Type, Temperature Condition and Sample Position.

Groups	Count	Mean (GPa)*	Variance(GPa^2)
Limestone, hot, top	5	31.90	30.59
Limestone, hot, mid	4	28.18	3.109
Limestone, cold, top	2	30.05	0.180
Limestone, cold, mid	2	28.39	5.712
River Rock, hot, top	2	37.34	30.109
River Rock, hot, mid	2	30.13	39.61
River Rock, cold, top	4	34.23	2.464
River Rock, cold, mid	4	33.40	3.618
All, hot, top	7	30.17	168.0
All, hot, mid	6	28.83	10.81
All, cold, top	6	35.53	24.87
All, cold, mid	6	30.30	7.722
All, top	12	33.94	23.77
All, mid	13	29.56	9.015

* 1 GPa = 145000 Psi

Based on the provided concrete samples, the data suggest that the observed differences in dynamic E at top and mid sections of the core samples are not caused by plastic concrete temperature conditions. The distributed microscopic damage state caused by high concrete plastic temperatures, within those samples, is not notably different at the top and bottom sections.
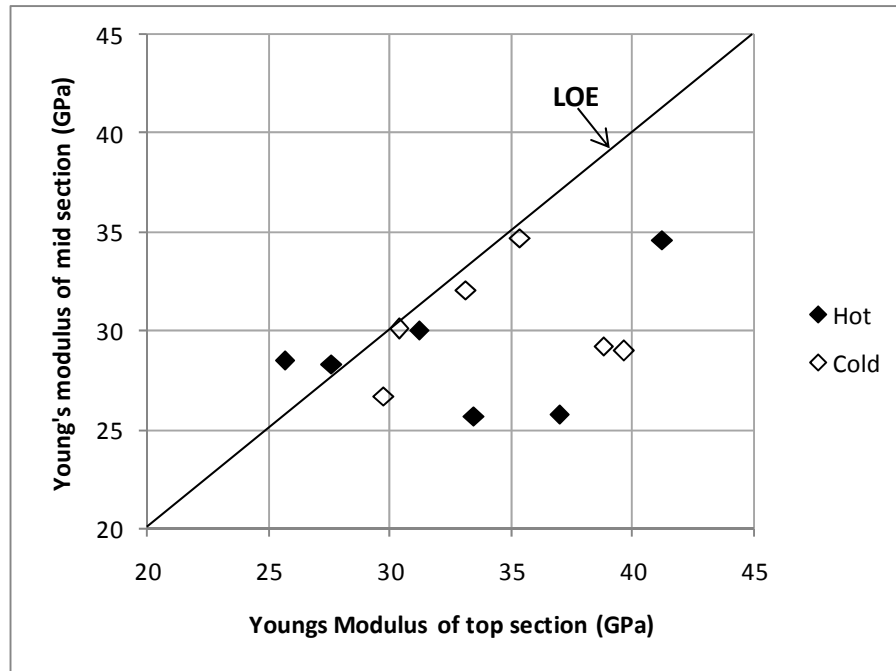
Figure 21. Comparison of dynamic Young's modulus from top and mid sections of same cores: comparison with regard to plastic temperature across all aggregate types.

5.1.4 Density, Absorption, and Void Content Tests

The standard test for density, absorption, and voids, known as the "boil" test, determines the bulk density, absorption, and the volume percentage of permeable pore space (ASTM C 642). The tests were performed on two sections of each core: one at the top of the core (near the pavement surface) and one directly under it. This was done in the hopes of showing that excessive drying or poor finishing techniques were causing an altered pore structure at the top of concrete poured in hot weather.

Testing Procedure

After completion of the resonance testing on the 5 cm core sections, they were oven-dried for successive 24-hr periods until no additional mass of moisture was lost. After recording the final oven-dry mass of each sample, the core sections were soaked in water at room temperature for successive 48-hr periods until no more mass of water was absorbed. Finally, after recording the saturated surface dry mass of each sample, the core sections were placed in a metal container, covered with water, and boiled for 5 hours, then allowed to cool in the water for at least 14 hours. The final boiled and surface dried sample weights were recorded, and the apparent mass of each sample was determined by measuring its mass suspended in water. These masses were used to calculate the data shown in Table 8, as prescribed by ASTM C 642.

Results

Table 8 shows the results from the boil test for the core sections followed by a statistical analysis of the obtained data.

Table 8. Boil Test Results for Core Samples

Label	Absorption after immersion (%)	Absorption after immersion and boiling (%)	Bulk density, dry (g/cc)	Bulk density after immersion (g/cc)	Bulk density after immersion and boiling (g/cc)	Apparent density (g/cc)	Volume of permeable pore space (%)
150D1-TOP	5.77%	5.89%	2.21	2.34	2.34	2.54	13.02%
150D1-MID	5.81%	5.78%	2.22	2.35	2.35	2.55	12.83%
150D2-TOP	7.16%	7.19%	2.17	2.33	2.33	2.58	15.62%
150D2-MID	7.09%	7.13%	2.17	2.33	2.33	2.57	15.48%
150R1-TOP	5.40%	5.49%	2.30	2.42	2.42	2.63	12.60%
150R1-MID	5.18%	5.35%	2.30	2.42	2.43	2.63	12.33%
150R2-TOP	5.16%	5.20%	2.31	2.43	2.43	2.63	12.02%
150R2-MID	5.44%	5.57%	2.29	2.41	2.41	2.62	12.73%
74R1-TOP	5.49%	5.66%	2.23	2.35	2.35	2.55	12.60%
74R1-MID	5.81%	6.17%	2.19	2.32	2.32	2.53	13.49%
74R2-TOP	5.09%	5.29%	2.26	2.37	2.38	2.57	11.95%
74R3-TOP	5.22%	5.40%	2.24	2.36	2.37	2.55	12.11%
74R3-MID	5.49%	5.73%	2.22	2.34	2.34	2.54	12.70%
74S1-TOP	4.91%	5.35%	2.16	2.27	2.28	2.44	11.58%
74S1-MID	4.72%	5.03%	2.14	2.24	2.25	2.40	10.75%
74S2-TOP	4.48%	4.72%	2.21	2.31	2.32	2.47	10.45%
74S2-MID	4.51%	4.83%	2.18	2.27	2.28	2.43	10.50%
CREB1-TOP	5.13%	5.26%	2.29	2.40	2.41	2.60	12.04%
CREB1-MID	5.42%	5.66%	2.26	2.39	2.39	2.60	12.81%
CREB2-TOP	4.97%	5.23%	2.30	2.41	2.42	2.61	12.01%
CREB2-MID	5.23%	5.47%	2.27	2.39	2.40	2.60	12.44%
CRWB1-TOP	4.93%	5.12%	2.35	2.46	2.47	2.67	12.02%
CRWB1-MID	5.05%	6.36%	2.27	2.38	2.41	2.65	14.43%
CRWB2-TOP	4.96%	5.13%	2.34	2.46	2.46	2.66	12.02%
CRWB2-MID	5.12%	5.89%	2.29	2.41	2.43	2.65	13.48%

The data were analyzed using the same statistical T test, described before. As before, we will assume a 95% confidence level, meaning that α=0.025.

A statistical analysis of the influence of aggregate type, plastic concrete temperature, and core sample section was carried out. The statistical data are presented in Table 9. A first look at the data in Table 9 suggests that the hot plastic concrete samples show higher permeable void space than the cold plastic concrete samples. However, this inference cannot be established at the 95% confidence level based on the T test results. This inference can be established at the 90% confidence level, though. The influence of aggregate type and core sample section on the permeable void space cannot be established with any meaningful statistical confidence. As before, the samples from the apparently undamaged sections of Cruger Road are considered to be "cold" for the purpose of this analysis, even though technically they did exhibit high plastic temperatures.

Table 9. Statistical Results of Permeable Pore Space Values Obtained from 5cm Core Samples, with Regard to Aggregate Type, Temperature Condition, and Core Section.

Groups	Count	Mean (%)	Variance(%^{2})
All, limestone	13	12.55	2.713
All, river rock	12	12.58	0.540
All, hot	13	13.01	1.474
All, cold	12	12.08	1.423
All, top	13	12.31	1.350
All, mid	12	12.83	1.874

5.1.5 Summary of Tests on Field Samples

Laboratory tests that are sensitive to distributed damage content (ASTM C 215 resonance frequency test) and permeable void volume (ASTM C 642 boil test) were carried out to determine if statistically significant differences are seen between concrete samples from "hot" and "cold" plastic temperature casting sites. The tests were also carried out on halved core samples to determine if property differences at the top surface and mid depth of the pavement are seen. Based on a statistical analysis of test results obtained from the provided concrete samples, data show that meaningful differences – with a statistical confidence of 95% - between hot and cold temperature cast sites are not seen in the dynamic modulus and permeable void volume data. Furthermore, statistically significant differences between the top and mid sections of the core samples owing to the temperature of the plastic concrete are not seen. However, we point out that the data suggest hot samples tend to show higher permeable void volume and lower dynamic modulus than the cold samples as a whole, but this cannot be established with a high degree of statistical confidence, i.e. 95%. Ultimately we conclude that the distributed damage state and permeable pore structure of concrete at the micro- and meso-scales is not significantly affected by concrete plastic temperatures within this sample set, although there is evidence of some moderate effects.

Considering the above mentioned problems caused by high weather concreting and the results presented in this report (some evidence of moderate damage and increased porosity caused by high plastic concrete temperatures), the authors suggest further testing to understand the behavior of concrete at high placement temperatures. The results may enable improved specification language to encourage the contractor to innovate and use improved cementitious materials at times when air and concrete temperatures are expected to exceed 90°F (32.2°C).

5.2 TESTS ON LABORATORY SAMPLES

A range of concrete mixtures were subjected to an extended high temperature mixing procedure. The mixtures contained different amounts and types of mineral and chemical admixtures. The performance of the fresh mixtures was evaluated by slump and air content tests. The compressive and split tensile strength of samples cast from the mixtures

were also measured. The tests evaluate the ability of a mixture to maintain suitable characteristics under a high temperature mixing and placement environment. Tests to measure early-age cracking tendency (e.g. the restrained ring shrinkage test) and long-term material durability performance (e.g. freezing and thawing resistance and scaling) of the material were not carried out here. These issues should be studied to complete a comprehensive evaluation of the behavior of concrete subjected to high temperatures. The restrained ring tests are especially important for understanding of concrete cracking behavior. A review of the current state of restrained ring shrinkage tests is provided in Appendix E.

5.2.1 Concrete Mixtures

Concrete mixtures were designed to consider various combinations of mineral and chemical admixtures. In all mixtures, Type I Portland cement, a natural quartz sand and crushed limestone coarse aggregate with CA 7 gradation were used. The plain mixture, without any mineral admixtures, satisfies Illinois DOT specifications for "PV" (pavement) concrete. Mixtures with varying levels of Class C fly ash and ground granulated blast-furnace slag, based on the plain mixture proportions, were designed to replace cement by mass. The component proportions of the mixtures are given in Table 10, and the dosages and detail on the admixtures are given in Table 11, unless specifically indicated in the footnotes in Tables 12 and 14. A given mixture contains all three types of chemical admixture: air-entraining admixture, superplasticizer and retarder at the listed dosages. In some cases however a mixture may contain only one or two chemical admixtures as indicated.

Table 10. Concrete Mixture Proportions Expressed (a) as lb/yd^3 (SSD) (b) as kg/m^3 (SSD)

Mixture	Plain	FA 1 Class C (20%)	FA 2 Class C (40%)	Slag 1 (20%)	Slag 2 (50%)	Ternary Class C, GGBS (25%-25%)
water	254.1	254.1	254.1	254.1	254.1	254.1
cement	605	484	363	484	302.5	302.5
sand	1131	1130	1121	1137	1132	1123
CA 7	1839	1836	1820	1848	1840	1825
flyash	0	121	242	0	0	151.25
slag	0	0	0	121	302.5	151.25

(a)

Mixture	Plain	FA 1 Class C (20%)	FA 2 Class C (40%)	Slag 1 (20%)	Slag 2 (50%)	Ternary Class C, GGBS (25%-25%)
water	150.8	150.8	150.8	150.8	150.8	150.8
cement	359.0	287.2	215.4	287.2	179.5	179.5
sand	671.2	670.6	665.3	674.8	671.8	666.5
CA 7	1091.4	1089.6	1080.1	1096.7	1092.0	1083.1
flyash	0	71.8	143.6	0	0	89.8
slag	0	0	0	71.8	179.5	89.8

(b)

Table 11. Description of Chemical and Mineral Admixtures Used

Admixture	Description	Dosage (ml / 100 kg cement)	Dosage (fl oz/ 100 lb cement)
Air entrainer	ASTM C 260 (Grace Daravair 1400)	80	1.22
Super-plasticizer	ASTM C 494 Types A&F/ASTM C 1017 Type I (Grace ADVA Cast 575)	107	1.64
Retarder	ASTM C 494 Type D (Grace Daratard 17)	340	5.20
Fly ash	ASTM C 618 Class C (Lafarge)	----	
Slag	ASTM C 989 grade 100 (Prairie Materials)	----	

5.2.2 Mixing and Test Procedures

Two concrete mixing procedures were employed: a standard procedure and an extended high temperature mixing procedure. The standard procedure follows the standard practice described by ASTM C 192. The extended high temperature mixing procedure, developed by the Florida Department of Transportation (FDOT 2009, Chapter 9.2.7), was employed to evaluate the performance of the concrete mixtures. Full detail of the current Florida hot weather concrete specification and hot weather mixture verification procedure is given in Appendix B, but the procedure is summarized here. The area around the mixture is held at a constant air temperature between 94 to 98°F (34.4 to 36.6°C). First, all dry mixture ingredients are pre-heated in this environment for several hours. Then the dry components are mixed together in a 2.5 cu.ft. (0.0707 m^3) capacity pan mixer for five minutes (three-minute initial mixing, three-minute rest, and two-minute follow up mixing) with hot water at 94°F (34.4°C). The slump (ASTM C 143 procedure) and air content (ASTM C 231 procedure) of the mixture immediately after the preliminary mixing process are measured. The remaining concrete in the mixer is then mixed for 30 seconds every five minutes while held at 94 to 98°F (34.4 to 36.6°C) for 90 minutes. The objective of this process is to ensure that the plastic concrete maintains a temperature of at least 94°F (34.4°C) throughout the process. During the rest periods, the mixture is covered to minimize moisture loss. After 90 minutes, the concrete slump and air content are measured again. In most cases, 4 x 8 inch (102 x 203 mm) cylinder samples of the concrete were made for subsequent strength tests. The cylinder samples were kept in the 94 to 98°F (34.4 to 36.6°C) for 24 hours, after which they were demolded and transferred to a room with a maintained environment of 73°F (22.7°C) and 100% humidity until the time of the strength tests. Standard compressive and split tensile strengths, ASTM C 39 and C 496 procedures, respectively, were measured at ages of 3, 7, and 14 days. For each strength type and age, the average value from three samples was reported.

Controlled air temperature in an area around the pan mixer was maintained by an enclosed thermally insulated area, which includes enough space to carry out all fresh concrete tests, as shown in Figure 22. Three portable space heaters were used to maintain an air temperature between 94 to 98°F (34.4 to 36.6°C) within the enclosed area. The air temperature was monitored using a multi-sensor system distributed within the enclosed space, and the average temperature was recorded. The temperature of the plastic concrete mixture was also measured at the end of the extended mixing procedure.

Figure 22. Testing set-up used for the high temperature extending mixing procedure: pan mixer and air temperature sensor control system (left) , enclosed testing area around mixture showing space heaters and fresh concrete test equipment (right).

Table 12. Fresh Concrete Test Results for Concrete using Extended High Temperature Mixing Procedure

Mix number	Mixture type	Slump before (inch/ mm)	Slump after (inch/ mm)	Air before (%)	Air after (%)	Slump loss (inch/ mm)	Air loss (%)	Mix Temp (°F)[3]
1	Plain[1], no retarder	5.0 / 127	0	4.0	2.7	complete	1.3	101.5
2	Plain, no retarder	5.5 / 139.7	0	6.8	2.8	complete	4.0	100.2
3	Plain[2]	7.5 / 190.5	1.5 / 38.1	7.3	7.0	6.0 / 152.4	0.3	95.7
4	Plain	8.0 / 203.2	2.0 / 50.8	5.0	7.7	6.0 / 152.4	-2.7	98.0
5	FA1[2] Class C	9.25 / 234.9	3.0 / 76.2	4.4	8.3	6.25 / 158.8	-3.9	95.6
6	FA1 Class C	9.0 / 228.6	3.0 / 76.2	3.3	8.0	6.0 / 152.4	-4.7	95.7
7	FA2 Class C	9.0 / 228.6	3.0 / 76.2	3.0	7.1	6.0 / 152.4	-4.1	97.9
8	Slag1	7.5 / 190.5	2.25 / 57.2	5.8	7.2	5.25 / 133.4	-1.4	95.4
9	Slag2	7.3 / 185.4	3.25 / 82.6	4.4	6.5	4.25 / 108	-2.1	96.5
10	Slag2 no retarder	7.0 / 177.8	0.5 / 12.7	4.7	2.7	6.5 / 165.1	2.0	100.0
11	Slag2 no SuperP	6.0 / 152.4	1.5 / 38.1	7.8	4.7	4.5 / 114.3	3.1	96.5
12	Ternery Class C and GGBS no SuperP	8.75 / 222.3	2.25 / 57.2	6.8	4.6	6.5 / 165.1	2.2	97.2

[1] Air = 29 ml/ckg (0.44 fl. oz/100 lb); SuperP = 54 ml/ckg (0.83 fl oz/100 lb)
[2] Air = 107 ml/ckg (1.64 fl oz/100 lb)
[3] °C = (°F – 32) x 5/9

Table 13. Strength Test Results for Samples using Extended High Temperature Mixing Procedure

Mix number	3-day comp. (psi*)	7-day comp. (psi)	14-day comp. (psi)	3-day split tensile (psi)	7-day split tensile (psi)	14-day split tensile (psi)
1	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*
2	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*
3	3600	3690	4635	344	448	571
4	3080	3525	3950	391	425	444
5	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*	*n/a*
6	2260	3005	3595	249	316	432
7	2090	2915	3880	280	383	437
8	3235	3635	5255	287	429	432
9	2455	3685	4230	262	398	405
10	n/a	n/a	n/a	n/a	n/a	n/a
11	2670	3685	5305	227	430	438
12	2640	3405	5185	200	422	380

* 1 psi = 6.89 kPa = 0.00689MPa

Table 14. Strength Test Results for Samples Using Standard Mixing Procedure (ASTM C 192) At 70°F (21.1°C)

Mix number	3-day comp. (psi*)	14-day comp. (psi)	28-day comp. (psi)
1[1]	3287	4181	4782
5[2] Type C	2744	4531	5088
5[3] Type F	2332	3258	4067
7[4]	2005	3776	4533
8[5]	2429	4070	4746
10[6]	1583	3882	4760

[1] Air = 57 ml/ckg (0.87 fl oz/100 lb); SuperP = 104 ml/ckg (1.59 fl oz /100 lb)
[2] Air = 73 ml/ckg (1.12 fl oz); SuperP = 41 ml/ckg (0.63 fl oz /100 lb)
[3] Air = 28 ml/ckg (0.43 fl oz /100 lb)
[4] Air = 103 ml/ckg (1.58 fl oz /100 lb)
[5] Air = 33 ml/ckg (0.50 fl oz/100 lb); SuperP = 19 ml/ckg (0.29 fl oz /100 lb); Retarder = 0 ml
[6] Air = 33 ml/ckg (0.50 fl oz /100 lb); SuperP = 38 ml/ckg (0.58 fl oz /100 lb); Retarder = 0 ml
* 1 psi = 6.89 kPa = 0.00689MPa

5.2.3 Laboratory Test Results

The high temperature extended mixing procedure developed by Florida DOT was successfully applied to several different concrete mixtures with varying mineral and chemical admixture content. Slump and air content were measured before and after the extended procedure for each mixture, and strengths of companion samples were measured at 3, 7, and 14 days. According to Illinois DOT specifications for "PV" (pavement) concrete, slump should be between 2 to 4 inches (50.8 to 101.6 mm), corrected air content between 5 to 8% and 14-day compressive strength should be above 3500 psi. As shown in Table 13, all tested concrete mixtures with mineral (class C fly ash and ground granulated blast-furnace slag) and chemical (air entraining admixture, superplasticizer, and retarder) admixtures met the minimum 14-day compressive strength requirement for "PV" concrete specification.

The fresh concrete properties for the high temperature mixing procedure are listed in Table 12. As expected, all mixtures suffered considerable slump loss during the extended, high temperature mixing procedure, which can be seen in Figure 23. It should be noted that the effective working time of the superplasticizer is not known. Since it is possible to meet the target slump at the end of the mixing process by beginning with a high slump in anticipation of the slump loss, a more effective evaluation process may be to identify mixtures that exhibit the smallest slump loss as a result of the extended mixing procedure. The addition of retarding admixture (ASTM C 494 Type D) is critical for minimizing slump loss and extending placement and finishing time of concrete mixtures exposed to extended high temperatures. Mixtures without retarding admixture suffered the highest levels of slump loss, and in some cases all slump was lost. This slump loss is likely due in part to accelerated hydration processes since it is clear that mixtures without retarding admixture exhibited the highest mixture temperature after the extended procedure (see Table 12).

Together with retarding admixture (ASTM C 494 Type D), high percentage replacement of cement with granulated blast furnace slag (ASTM C 989 grade 100) significantly reduced the slump loss of the concrete for the material combinations tested (see Figure 23) and resulted in the highest compressive strength at 14 days. The mixtures with fly ash (ASTM C 618 Class C) partially replacing cement were able to achieve the target placement slump at the end of the mixing period as long as the initial slumps were high enough. However, the fly ash based mixtures that included even a retarding admixture (ASTM C 494 Type D) did not affect the slump loss rate of concrete mixtures like the slag mixtures. This behavior was seen in a recent study with recycled concrete aggregates and may be specific to the particular Class C fly ash being used in the laboratory.

The air content was a difficult property to control in the concrete laboratory mixtures as seen in Table 12. Mixtures that contained both the superplasticizer and retarding admixture showed an increase in air content as a result of the extended mixing procedure. This was a result of unintended chemical interaction between these two admixture types. It can be seen that multiple mixtures had negative admixture interactions which increased the air content at the end of the mixing time. The fly ash mixtures appeared to be most affected by this interaction. When retarder and superplasticizer were not used simultaneously in a mixture, the air content in the mixture decreased with mixing time as expected. A similar negative interaction between certain Class C fly ash, certain water reducers, and cement types has been reported by Kohn and Tayabji (2003) for hot temperature paving mixtures. They also suggested testing the proposed mixtures at higher temperatures to determine if a negative interaction exists. If it does, changing the water reducer type, cement source, or fly ash source should rectify the problem.

To verify that the high temperature mixtures didn't have adverse strength that would affect long-term performance, several of the mixtures were mixed at 70°F (21.1°C) as shown

in Table 14. All these mixtures had an approximate initial air content of 6 percent and slump of 6 inches (152.4 mm). The air entraining agent and superplasticizer was varied to achieve these targets and retarder was not added at this mixing temperature. These batches didn't follow the 90 minute mixing procedure and the specimens were cast immediately. All the mixtures in Table 14 met the minimum compressive strength of 3,500 psi at 14 days except mixture 5 with Class F fly ash. When comparing similar mixtures in Table 13 (high temperature conditions) with Table 14, the strengths are consistently higher at 3 and 14 days for extended mixing. This can be attributed to better mixing and uniformity afforded by the addition of superplasticizer and retarder, and also that the higher initial mixture temperature promotes higher early strength gain. The strength results at 70°F (21.1°C) confirm the reasonableness of the high temperature mixture strategies and data presented in Table 13.
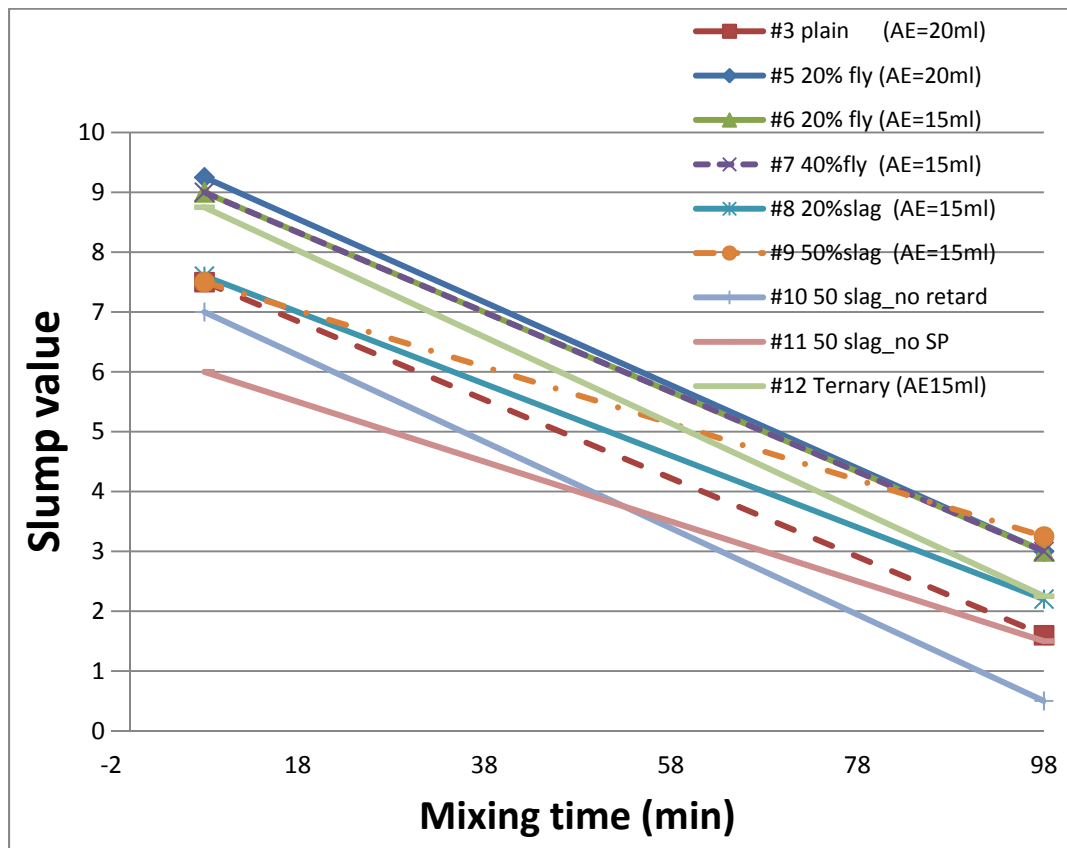
Figure 23. Slump loss versus high temperature mixing time for various concrete mixtures. AE = air-entraining admixture, SP = Superplasticizer.

5.2.4 Summary of Laboratory Results

The high temperature concrete mixing procedure used by the Florida DOT shows merit and potential for evaluating the effectiveness of certain mixture changes on the fresh and hardened properties of the concrete. This process demonstrated that concrete mixtures that are not altered during high temperature application will certainly lead to potential problems with placement, compaction, and finishing, and in all likelihood, future durability issues also. For this study, the FDOT procedure identified the combination of material

constituents that were able to achieve workable, compactable, and finishable concrete that meets IDOT´s specifications. It also was able to indicate potential interactions that could result in problems in the initial construction or long-term durability.

CHAPTER 6 CONCLUSIONS

Based on the results presented in this report, the following conclusions are drawn:

6.1 SURVEY OF PRACTICE

To obtain information on states' practices for hot weather concreting, a survey was sent by email to appropriate materials contacts in each state department of transportation. The survey asked about the maximum allowable plastic concrete temperatures for pavements and structures and asked if there was any deterioration that was attributed to paving in hot weather. The answers are summarized as follows:

- Most states specify a maximum allowable plastic concrete temperature of 90°F (32.2°C) or lower. However, the time at which this temperature must be taken, for example at delivery or before final placement, is not always clearly specified.
- Illinois is the only state, to our knowledge, that specifies two separate maximum plastic concrete temperatures, one at delivery before placement (90°F = 32.2°C) and one as placed at 96°F (35.5°C).
- Several states, including nearly all of the neighboring states to Illinois, do not specify a maximum allowable temperature for plastic concrete. Most often, these states employ end-result specifications. This approach places the burden on the contractor, but also allows for contractor flexibility to achieve the quality end product with a range of methods.
- With regard to concrete damage thought to be caused by high mixture temperatures, most states that responded to the questionnaire report no known temperature-related distress. Of the states that do report damage, most is in the form of shrinkage or structural cracking.
- From the limited survey sample, the results suggest that the states that have a maximum allowable limit of 90°F (32.2°C) or lower control the hot weather concreting problem fairly well, as these states also report low occurrence of temperature-related distress.
- Many states specify other practices, such as required use of retarder, fly ash, haul time limits, etc., in addition to maximum allowable plastic concrete temperatures. Several states claim that incorporation of mineral admixtures provides a cost-effective alternative to other heat mitigating actions such as mixing with ice or on-site water cooling.

6.2 ILLINOIS SITE VISITS

The investigating team conducted three visits to roadway sites in Illinois with suspected hot weather concrete problems. The three site locations visited were in Chicago, Peoria (three projects within Peoria were seen on the same day), and Collinsville, Illinois. These projects include major interstate highway pavement and a barrier wall, interstate exit ramps, local roads, and a driveway.

- The concrete at the three sites visited did exhibit some combination of excessive cracking, rapid slump loss, early setting time, and poor finishability. The common feature in these three sites is that concrete temperatures were high, either very close to or above 90°F (32.2°C). However, the overall extent of this problem in Illinois is not clear.

- All visited sites were slip-formed projects and thus the concrete had a low water to cementitious materials ratio. It is possible that these concrete mixtures showed increased susceptibility to hot weather concreting problems due to challenges in placing, consolidating, and finishing low slump concrete.

6.3 EXPERIMENTAL TESTS ON FIELD SAMPLES

Core samples were obtained from one of the site visit locations (Peoria) for the purpose of performing a series of laboratory tests. The primary goal of the tests was to determine what differences (if any) were present between concrete poured in hot weather conditions and those poured under normal temperatures. The field samples were analyzed by standard resonance frequency analysis, the standard test for density, absorption, and voids, i.e. the "boil" test.

- Based on a 95 percent statistical confidence level, the test results obtained from the provided field concrete samples show no meaningful differences between hot and cooler temperature cast sites in terms of both dynamic modulus and permeable void volume data. Furthermore, statistically significant differences between the top and mid sections of the field core samples, owing to the temperature variation with depth of the plastic concrete, are not seen.
- Although the data suggest "hot" samples tend to show higher permeable void volume and lower dynamic modulus than the "cold" samples as a whole, this cannot be established with a high degree of statistical confidence, i.e., 95%.
- Based on these data from the test site samples, we conclude that the distributed damage state and permeable pore structure of concrete at the micro- and meso-scales is not significantly affected by concrete plastic temperatures within this sample set.

6.4 EXPERIMENTAL TESTS ON LABORATORY SAMPLES

Fresh concrete mixtures were subjected to the high temperature mixing procedure developed by Florida DOT. Air content and slump were measured before and after the 90-minute mixing procedure as well as compressive and splitting tensile strengths at various ages. The principal goals of the tests were to determine the effects of mineral and chemical admixtures on the performance of fresh concrete at high temperatures and to meet the minimum strength at 14 days. Tests to measure early-age cracking tendency (e.g. the restrained ring shrinkage test) and long-term material durability performance (e.g. freezing and thawing resistance and scaling) of the material were not carried out here, but still need to be conducted.

- At the end of the 90 minute high temperature mixing procedure, "PV" concrete without retarder, Superplasticizer (high-range water-reducing admixture), or mineral admixtures was not able to meet the specified fresh concrete properties.
- All tested concrete mixtures with mineral (Class C fly ash and ground granulated blast-furnace slag) and chemical (air entraining admixture, superplasticizer, and retarder) admixtures met the 14-day strength requirement for "PV" concrete.
- The addition of retarding admixture (ASTM C 494 Type D) is critical for minimizing slump loss and entrained air loss and extending placement and

finishing time of concrete mixtures exposed to extended high temperatures during mixing, placement, and finishing.

- Together with retarding admixture (ASTM C 494 Type D), high percentage replacement of cement with ground granulated blast-furnace slag (ASTM C 989 grade 100) significantly reduced the concrete slump loss over the mixing time for the material combinations tested.
- Together with retarding admixture (ASTM C 494 Type D), cement replacement with fly ash (ASTM C 618 Class C) did not notably affect the slump loss rate of concrete mixtures exposed to extended high temperature mixing. However, this finding may be specific to the particular fly ash that was utilized.

CHAPTER 7 RECOMMENDATIONS

The research did not show systematic evidence of frequent cracking problems in Illinois concrete projects related to high temperature placement. However, there appears to be occasional placement, consolidating, and finishing issues with concrete when concrete temperatures approach or exceed 90°F (32.2°C). Therefore, the authors recommend a change in specification language to reduce uncertainty and also encourage the contractor to innovate and use mineral admixtures at times when air and concrete temperatures are expected to exceed 90°F (32.2°C). These recommendations address the issues of slump loss and entrained air loss in the plastic mixtures, but they do not directly consider cracking potential and long-term durability.

The current wording of the Illinois DOT specification (Standard Specifications for Road and Bridge Construction Adopted January 1, 2007) for temperature control for placement is the following:

1020.05(b)(10)

> For Class PV, MS, and SI, a retarding admixture shall be added to the concrete mixture when the concrete temperature is 85 °F (30 °C) or higher. This requirement may be waived by the Engineer when fly ash compensated mixtures are used.

1020. 14 Temperature control for Placement. Temperature control for concrete placement shall be according to the following.

(a) Concrete other than Structure. Concrete may be placed when the air temperature is above 35 °F (2 °C) and rising, and concrete placement shall stop when the falling temperature reaches 40 °F (4 °C) or below, unless otherwise approved by the Engineer.

The temperature of concrete immediately before placement shall be a minimum of 50 °F (10 °C) and a maximum of 90 °F (32 °C). Plastic concrete temperature up to 96 °F (35 °C), as placed, may be permitted, provided jobsite conditions permit placement and finishing without excessive use of water on and/or overworking of the surface. The occurrence within 24 hours of unusual surface stresses shall be cause to revert to a maximum 90 °F (32 °C) plastic concrete temperature.

(c) All classes of concrete. Aggregates and water shall be heated or cooled as necessary to produce concrete within the specified temperature limits.

The authors recommend the following changes be made to this specification:

- Article 1020.14 (a): The dual-valued temperature specification (delivered and in-place) is unusual; Illinois is the only state to the authors' knowledge that specifies this. In order to reduce possible confusion, we suggest one value of specified temperature: as delivered at 90°F (32.2°C). Most states surveyed that had a 90°F (32.2°C) delivered specification did not report problems of damage caused by high placement temperatures. However to give flexibility to contractors, higher delivered temperatures, up to 96°F (35.5°C), could be allowed if the proposed mixtures meet existing IDOT performance targets for air, slump, and strength when using the FDOT high temperature mixing procedure.
- Article 1020.14 (c): We recommend replacing "shall" with "may" in reference to the sentence on aggregate and water heating/cooling. Appropriate incorporation of mineral admixtures may provide a cost-effective alternative to aggregate/water cooling activities.
- Article 1020.05 (b) (10): The authors recommend that the current recommendation for addition of retarder be maintained. Our data suggest that non-retarding water reducers (superplasticizer) alone are not sufficient to preserve slump loss and entrained air loss at high plastic temperatures. Therefore at temperatures higher than 85°F (29.4°C), some type of retarding admixture should be used.
- Article 1020.05 (b) (10): Also, the authors recommend removing the statement that waives the retarder requirement if fly ash is added. Although some fly ashes may work suitably for this purpose, the Class C fly ash that was tested did not improve the slump loss performance at high temperatures. So the original specification wording concerning fly ash is likely too broad.

Based on these recommendations, the revised specification would read as follows:

Portland cement concrete other than structures. The temperature of mixed concrete immediately before placing shall not be less than 10°C (50°F) nor more than 32°C (90°F). Aggregates and water may be heated or cooled as necessary to produce concrete within these temperature limits.
When the temperature of the plastic concrete reaches 30°C (85°F) an approved retarding admixture shall be used. Plastic concrete temperatures of up to 35°C (96°F) immediately before placing may be permitted if the specific mixture has been demonstrated to satisfy the FDOT high temperature mixing procedure test. Mineral admixtures may be used up to 50% cement replacement for these high temperature situations. If the mixture does not pass the FDOT high temperature mixing procedure test, then it is not approved for hot weather use, and work on the project must cease if the concrete temperatures before placement exceed 90°F (32.2°C).

REFERENCES

A Guide to Evaluating Thermal Effects in Concrete Pavements, Report SHRP-C-321. Strategic Highway Research Program, National Research Council, Washington, D.C., 1992.

American Concrete Institute (ACI), "ACI 305.1-06: Specifications for Hot Weather Concreting" American Concrete Institute, Farmington Hills, MI.

ACI Committee 305, Hot Weather Concreting, American Concreting Institute, Farmington Hills, Michigan, 1999.

Bergin, M, and C. Syed, "Florida's Hot-Weather Concreting Specification", Concrete International, May 2002, American Concrete Institute, Farmington Hills, MI.

Bilodeau A., and V.M. Malhotra, High Volume Fly Ash Systems, Concrete Solution for Sustainable Development, *ACI Materials Journal*, Vol. 97, No. 1, pp. 41-48, 2000.

IDOT. 2007. Standard Specifications for Road and Bridge Construction.

Kohn, S.D. and S.D. Tayabji, Best Practices for Airport Portland Cement Concrete Pavement Construction (Rigid Airport Pavement), Report IPRF-01-G-002-1, Innovative Pavement Research Foundation Program, Washington, DC, 2003.

Kjellsen, K. O., R. J. Detwiler, and O.E. Gjørv, Development of Microstructures in Plain Cement Pastes Hydrated at Different Temperatures, Cement and Concrete Research, Vol. 21, No. 1, pp. 179-189, 1991.

Ma W, Sample D., R. Martin, and P.W. Brown, Calorimetric Study of Cements Blends Containing Fly Ash, Silica Fume and Slag at Elevated Temperatures, Cement Concrete and Aggregates, Vol. 16, No. 2, pp. 93-99, 1994.

Manmohan D., and P.K. Mehta, Heavily Reinforced Shear Walls and Mass Foundations Built with Green Concrete, *Concrete International*, Vol. 24, No. 8, pp. 64-70, 2002.

McCullough B. F., and T. Dossey, Evaluation of the Performance of Texas Pavements Made with Different Coarse Aggregates. Research Report 3925-1F. Center for Transportation Research, University of Texas at Austin. 1999.

Mehta P. K. and W. Langley, Monolith Concrete Foundation: Built to Last a 1000 Years, Concrete International, Vol. 22, No. 7, pp. 27-32, 2000.

Mehta P. K. and P.J Monteiro, Concrete: Microstructure, Properties, and Materials, McGraw Hill, College Custom Series, 550 p., 1993.

Mehta P. K., Performance of High Volume Fly Ash Concrete in Hot Weather, Innovations in Design with Emphasis on Seismic, Wind, and Environmental Loading; Quality Control and Innovations in Materials / Hot Weather Concreting, Proceedings, ACI Fifth International Conference, Cancun Mexico, 2002.

Neville, A., *Properties of Concrete,* 4th Edition, John Wiley and Sons, Inc., New York, 1996.

Samarai, M., S. Popovics, and V.M. Malhotra, "Effect of High Temperatures on the Properties of Fresh Concrete," Transportation Research Record, 924, pp. 42-50.

Schindler A. and B.F. McCullough, "Importance of Concrete Temperature Control During Concrete Pavement Construction in Hot Weather Conditions," Transportation Research Record, 1813, pp. 3-10.

Verbeck G. J., and R.H. Helmuth, Structure and Physical Properties of Cement Pastes, Proceedings Fifth International Symposium on the Chemistry of Cement, Tokyo, Vol. III, pp. 1-32, 1968.

Wapole, R.E. *Introduction to Statistics*, 3rd Edition, MacMillian Publishing Company, New York. 1982.

APPENDIX A: IDOT CONCRETE TEMPERATURE SPECIFICATION

Excerpt taken from Illinois Department of Transportation Standard Specifications for Road and Bridge Construction, 2007

1020.14 Temperature Control for Placement.   Temperature control for concrete placement shall be according to the following.

(a) Concrete other than Structures.   Concrete may be placed when the air temperature is above 35 °F (2 °C) and rising, and concrete placement shall stop when the falling temperature reaches 40 °F (4 °C) or below, unless otherwise approved by the Engineer.

The temperature of concrete immediately before placement, shall be a minimum of 50 °F (10 °C) and a maximum of 90 °F (32 °C). Plastic concrete temperatures up to 96 °F (35 °C), as placed, may be permitted, provided jobsite conditions permit placement and finishing without excessive use of water on and/or overworking of the surface. The occurrence within 24 hours of unusual surface distress shall be cause to revert to a maximum 90 °F (32 °C) plastic concrete temperature.

APPENDIX B: FDOT HOT WEATHER CONCRETE SPECIFICATION

***Excerpt taken from Florida department of transportation specifications,
section 346: portland cement concrete***

346-7.5 Concreting in Hot Weather: Hot weather concreting is defined as the production, placing and curing of concrete when the concrete temperature at placing exceeds 85°F but is less than 100°F.

Unless the specified hot weather concreting measures are in effect, reject concrete exceeding 85°F at the time of placement. Regardless of special measures taken, reject concrete exceeding 100°F. Predict the concrete temperatures at placement time and implement hot weather measures to avoid production shutdown.

***Excerpt taken from Florida department of transportation materials manual,
Section 9.2, Volume II: concrete production facilities guide***

Concrete trial mix temperature of 94°F:

(1) Ensure that preparation and testing of the trial mixes is performed in accordance with **ASTM C192**, with the following exceptions.

(2) Initial mixing shall be done in accordance with **ASTM C192**, except concrete materials shall be brought to a temperature that will ensure the mix temperature is not less than 94°F at any time.

(3) Hold the trial mix in the mixer for 90 minutes after completion of initial mixing. During the extended mixing period, turn the drum intermittently for 30 seconds every five minutes. Cover the drum with wet burlap or an impermeable cover material during the rest periods. At the end of the 90-minute period, remix the trial mix for a minimum of one minute and make a slump test to verify that the concrete is within the specified range for slump. Ensure that the mix temperature is not less than 94°F at any time.

(4) On completion of the extended mixing period, ensure that the trial mix concrete has a slump within ±0.75 inch of the target value (±1.0 inch for mixes utilizing a High Range Water-Reducing admixture), and an air content between 2.0 percent and 5.0 percent. If below the target range, the producer may adjust the slump by a water addition. After the water addition, remix the concrete for a minimum of two minutes.

(5) The total water used in initial mixing and the final slump adjustment constitutes the design mix water content. Ensure that the total water to cementitious materials ratio does not exceed the maximum water to cementitious materials ratio in the **Florida Department of Transportation Specifications, Section 346**, for the respective class of concrete. The Department may require extended mixing for precast/prestressed concrete when centrally mixed at the placement site.

APPENDIX C: UNIVERSITY OF ILLINOIS SURVEY

<u>Respondent information:</u>

State: _____   Name: _____   Position: _____

Follow up contact info: phone number: _____   email address: _____

<u>Survey questions:</u>

In your state, what is the maximum allowable temperature for concrete placement:

For pavements? _____   For structures or mass concrete? _____

In your state, are there any local variations in policy (i.e. by district)? ☐ Yes   ☐ No

If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? ☐ Yes   ☐ No

If yes:

What type and severity of deterioration? _____

What is the expected reason that the concrete was hot?

       ☐ High ambient temperature

       ☐ High concrete temperature upon arrival

       ☐ Other

    If other, please explain: _____

       Where is the damage located?

       ☐ Pavement   ☐ Bridge decks ☐ Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) _____

In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used. ☐ Yes   ☐ No

If yes, please explain: _____

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? _____

Any additional comments regarding hot weather concrete placement? _____

This questionnaire is part of a research project conducted by the University of Illinois for the Illinois Department of Transportation. Your cooperation in this survey is very much appreciated, and a copy of the results will be provided if you request it.

Forms can be filled out electronically in Microsoft Word and emailed to cpeters3@uiuc.edu, or returned via fax to (217) 265-8040 (Attn: John Popovics). We ask that you return the completed survey by March 31, 2008. Please feel free to contact us with any questions:

Dr. John Popovics, (217) 244-0843, johnpop@uiuc.edu
Dr. Jeff Roesler, (217) 265-0218, jroesler@uiuc.edu
Carrie Peterson, (217) 333-1773, cpeters3@uiuc.edu

APPENDIX D: SURVEY RESPONSES

Respondent information:
State: <u>AL</u>   Name: <u>Sergio Rodriguez</u>   Position: <u>Concrete Engineer</u>
Follow up contact info:
phone number: <u>(334)206-2411</u>   email address: <u>rodriguezs@dot.state.al.us</u>

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
for pavements? <u>90°F (32.2°C)</u>
For structures or mass concrete? <u>90°F (32.2°C) for bridge decks and 95 °F for other</u>
<u>structures</u>
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   X No
If yes, please explain: <u>NA</u>

Is there any deterioration in concrete attributed to high plastic temperatures?
        X Yes ☐No
If yes:
What type and severity of deterioration? <u>Shrinkage cracks - minor to moderate</u>
What is the expected reason that the concrete was hot?
        X High ambient temperature
        X High concrete temperature upon arrival
        ☐Other
        If other, please explain: _____

        Where is the damage located?
        X Pavement   X Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) <u>Acceptable practices are to cool</u>
<u>aggregates or to use ice in the mix as part of mixing water</u>
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
X Yes ☐No
If yes, please explain: <u>Approved retarder admixture must be used between June 1st and</u>
<u>September 15th and any other time when the temperature of the concrete may be above of</u>
<u>the maximum allowed.</u>

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? <u>Contractor assumes cost for cooling</u>
<u>practices</u>

Any additional comments regarding hot weather concrete placement? <u>ALDOT also</u>
<u>experienced problems on one bridge deck due to the Coefficient of Thermal Expansion of</u>
<u>the coarse aggregate. There is an ongoing research to determine if this problem is affecting</u>
<u>other structures. The problem with the CTE can be aggravated by high ambient</u>
<u>temperatures.</u>

<u>The following link will provide access to ALDOT specifications: ALDOT Specifications</u>

Respondent information:
State: <u>California</u>   Name: <u>Charles Dayton</u>   Position: <u>Senior Trans Engr</u>
Follow up contact info:
phone number: <u>(916) 227-5280</u>   email address: <u>charles_m_dayton@dot.ca.gov</u>

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? <u>90°F</u>
For structures or mass concrete? <u>Determined by Thermal Plan</u>
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   ☒No
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures?
           ☒Yes☐No
If yes:
What type and severity of deterioration? <u>Shrinkage Cracks</u>
What is the expected reason that the concrete was hot?
           ☒High ambient temperature
           ☒High concrete temperature upon arrival
           ☒Other
           If other, please explain: <u>Large Mass of Concrete</u>

           Where is the damage located?
           ☒Pavement   ☒Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) <u>All</u>
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☐Yes ☒No
If yes, please explain: _____

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? <u>Mitigation measures are stated in the Contract. Contractor pays mitigation as part of unit costs.</u>

Any additional comments regarding hot weather concrete placement? <u>Schedule concrete pours late in the afternoon so heat of hydration occurs at night with lower ambient temperatures.</u>

Respondent information:
State: Colorado   Name: Eric Prieve   Position: Concrete & Physical
Properties Support Engineer
Follow up contact info:
phone number: 303-398-6542   email address: Eric.Prieve@dot.state.co.us

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? 90F concrete temperature. No limit on air temperature
For structures or mass concrete? 90F concrete temperature. No limit on air temperature
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   ☒No
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures?
        ☐Yes☒No
If yes:
What type and severity of deterioration? _____
What is the expected reason that the concrete was hot?
        ☐High ambient temperature
        ☐High concrete temperature upon arrival
        ☐Other
        If other, please explain: _____

        Where is the damage located?
        ☐Pavement   ☐Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) Fogging placed concrete until curing material is in place. Night time placement.
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☐Yes ☒No
If yes, please explain: _____

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? Included in the contractor's price of materials.

Any additional comments regarding hot weather concrete placement? _____

Respondent information:
State: <u>CT</u>   Name: <u>Daniel E. Guzzo</u>   Position: <u>Materials Testing Engineer</u>
Follow up contact info:
phone number: <u>860.258.0339</u>   email address: d<u>aniel.guzzo@po.state.ct.us</u>

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? <u>90 degrees F</u>
For structures or mass concrete? <u>90 degrees F could be less if item is a special provision</u>
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   X No
If yes, please explain: <u>   </u>

Is there any deterioration in concrete attributed to high plastic temperatures?
          ☐Yes X No

If yes:
What type and severity of deterioration? <u>   </u>
What is the expected reason that the concrete was hot?
          ☐High ambient temperature
          ☐High concrete temperature upon arrival
          ☐Other
          If other, please explain: <u>   </u>

          Where is the damage located?
          ☐Pavement   ☐Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) <u>ice chips, spraying aggregate</u>
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☐Yes X No
If yes, please explain: <u>   </u>

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? <u>That cost is between the contractor and supplier.</u>

Any additional comments regarding hot weather concrete placement? <u>Contractor should schedule pours in early a.m. and work with supplier to achieve concrete temperatures within specification</u>

Respondent information:
State: Florida   Name: Mike Bergin   Position: State Structural Materials Eng.
Follow up contact info:
phone number: 352-955-6666   email address: michael.bergin@dot.state.fl.us

Survey questions:
          In your state, what is the maximum allowable temperature for concrete placement:
For pavements? 100F
For structures or mass concrete? currently 35 F teperature differential
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   ☒No
If yes, please explain: _____
Is there any deterioration in concrete attributed to high plastic temperatures?
          ☐Yes☒No
If yes:
What type and severity of deterioration? _____
What is the expected reason that the concrete was hot?
          ☐High ambient temperature
          ☐High concrete temperature upon arrival
          ☐Other
If other, please explain: _____

          Where is the damage located?
          ☐Pavement   ☐Bridge decks   ☐Secondary (e.g. barrier walls)

          In your state, what is the standard practice for placing concrete in high
temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) Contractor is required
to submit a mass concrete temperature control plan developed by a specialty engineer. This
is reviewed and approved by the Department. For elements or structures other than mass
concrete we do not allow concrete to be placed when the concrete temperature exceeds
100F.
In your state, are there any weather-related restrictions on mix design? For example, if
concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement
ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☒Yes ☐No
If yes, please explain: If the concrete temperature is expected to rise above 85F the
concrete producer is required to meet the hot weather concrete conditions. His trial batch is
held in the mixer for 90 minutes. During the 90 minute interval the batch is remixed for 30
seconds every 5 minutes. At the end of the 90 minute mixing cycle the concrete must be
within the minimum slump range allowed for that class of concrete. Fly ash and slag are
permitted and many of our producers use these to help control temperature and to reduce
cement content. Concrete cannot be placed if the temperaure of the concrete exceeds 100
F.

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? <u>The weather can be hot but he concrete must be cool for the contractor to place it! Mass concrete is described in the plans when the projects are bid. Our contractors know that a specialty engineer is required if mass concrete is noted in the plans and their bids typically reflect this. It cost the department more initally but the longer service life and durability is worth it.</u>

Any additional comments regarding hot weather concrete placement? <u>We've been placing concrete in hot weather for a couple of years and it can be done but the contractor must pay attention to the details including the curing of the elements. If you have an interest in our hot weather specifications for concrete please let me know.</u>

Respondent information:
State: GA   Name: Myron K. Banks   Position: M&R Branch Chief - Concrete
Follow up contact info:
phone number: 404-363-7561   email address: mbanks@dot.ga.gov

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? n/a
For structures or mass concrete? 90 (structures)
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   ☒No
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? ☒Yes   ☐ No
If yes:
What type and severity of deterioration? plastic shrinkage cracking/ minor
What is the expected reason that the concrete was hot?
     ☒High ambient temperature
     ☒High concrete temperature upon arrival
     ☒Other
     If other, please explain: hot cement

     Where is the damage located?
     ☐Pavement   ☒Bridge decks   ☒Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) ice chips, spraying, placement early in day or evening placements
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☐Yes ☒No
If yes, please explain: _____

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? contractor/supplier

Any additional comments regarding hot weather concrete placement? _____

Respondent information:
State: <u>Iowa</u>   Name: <u>Todd Hanson</u>   Position: <u>PCC Engineer</u>
Follow up contact info: phone number: <u>515-239-1226</u>
email address: <u>todd.hanson@dot.iowa.gov</u>

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? <u>n/a</u>
For structures or mass concrete? <u>90F</u>
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   x☐No
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? ☐Yes   x
☐No
If yes:
What type and severity of deterioration? _____
What is the expected reason that the concrete was hot?
          ☐High ambient temperature
          ☐High concrete temperature upon arrival
          ☐Other
          If other, please explain: _____

          Where is the damage located?
          ☐Pavement   ☐Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) <u>any method</u>
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☐Yes x☐No
If yes, please explain: we <u>use maturity method</u>

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? <u>contractor</u>

Any additional comments regarding hot weather concrete placement? _____

Respondent information:
State: <u>Illinois</u>   Name: <u>Doug Dirks</u>   Position: <u>Engineer of Concrete and Soils</u>
Follow up contact info:
phone number: <u>217-782-7208</u>   email address: <u>Douglas.Dirks@illinois.gov</u>

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? <u>96 degrees F</u>
For structures or mass concrete? <u>90 degrees F</u>
In your state, are there any local variations in policy (i.e. by district)? <u>Yes</u>
If yes, please explain: <u>District 4 limits pavement temperature to 90 degrees F.</u>

Is there any deterioration in concrete attributed to high plastic temperatures? <u>Yes</u>
If yes:
What type and severity of deterioration? <u>Deterioration consists of more cracking than
normal, pop outs, poor finishing, reduced tining, and excessive polishing. Deterioration is
not severe, but could eventually lead to distress in later years.</u>
What is the expected reason that the concrete was hot?
        <u>X</u> High ambient temperature
        <u>X</u> High concrete temperature upon arrival
        ☐Other
        If other, please explain: _____

        Where is the damage located?
        <u>X</u> Pavement   <u>X</u> Bridge decks   <u>X</u> Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g.
ice chips, spraying, ponding, liquid nitrogen, etc.) <u>Adding ice to the concrete mixture, wetting
concrete trucks, wetting aggregate stockpiles, and chillers to cool the concrete mix water.</u>
In your state, are there any weather-related restrictions on mix design? For example, if
concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement
ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
<u>No</u>
If yes, please explain: _____

How is compensation to contractors handled? For example, who absorbs the costs for
cooling concrete or lost time due to hot weather? <u>Contractor is responsible.</u>

Any additional comments regarding hot weather concrete placement? <u>No</u>

Respondent information:
State: Indiana   Name: Anthony Zander   Position: Concrete Engineer
Follow up contact info:
phone number: 317-610-7251 ext 234   email address: azander@indot.in.gov

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? none
For structures or mass concrete? none
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   ☒No
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? ☒Yes   ☐ No
If yes:
What type and severity of deterioration? cracks
What is the expected reason that the concrete was hot?
          ☒High ambient temperature
          ☒High concrete temperature upon arrival
          ☒Other
          If other, please explain: Concrete will likely not retain workablity (i.e. slump) in hot weather, so adding water becomes the quick fix, but reduces the quality of the concrete. However, without enough bleed water and poor curing practices, the evaporation rate can exceed what the concrete can provide so plastic shrinkage cracks can develop and evolve into full depth drying shrinkage cracks.

          Where is the damage located?
          ☐Pavement   ☒Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) no practice at this time
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used. ☒Yes ☐No
If yes, please explain: INDOT has required the application of evaporative retardant on select bridge decks that use concrete with low W/C (e.g. 0.420 or less),unless the Contractor can varify that the evaporation rate, based on ambient conditions of air temp, relative humidity, concrete temperature and wind speed, does not exceed the maximum allowable.

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? As far as INDOT is concerned, the Contractor always absorbs the cost of complying with all specifications since that is the party to which INDOT entered the contract document. The Contractor may very well pass it along to his mix producer, but that is not per an INDOT requirement.

Any additional comments regarding hot weather concrete placement? Since INDOT builds plain PCCP with contraction joints spaced at about 15' to 18', there are no major problems with damage or distress due to shrinkage. We may be getting distress due to the warpping or curling of the PCCP if the thickness of the PCCP gets too much.

Respondent information:
State: Kansas   Name: Dave meggers   Position: Research Development Engr.
Follow up contact info:
phone number: 785-291-3845   email address: Dave.Meggers@ksdot.org

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? N/A
For structures or mass concrete? N/A
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   ☒No
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? ☒Yes   ☐ No
If yes:
What type and severity of deterioration? Cracking has been noted on bridge decks.
What is the expected reason that the concrete was hot?
          ☒High ambient temperature
          ☒High concrete temperature upon arrival
          ☐Other
          If other, please explain: _____

          Where is the damage located?
          ☐Pavement   ☒Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) ice is most common or night placement
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☐Yes ☒No
If yes, please explain:
How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? Contractor

Any additional comments regarding hot weather concrete placement? 1. Maintain temperature of the mixed concrete between 50 to 90°F (32.2°C). at the time of placing 2. if the concrete temperature at time of placement is 90°F (32.2°C). or above, or under conditions contributing to quick stiffening of the concrete, place the concrete within 45 minutes of adding the cement to the water 3. Maintain environmental conditions on the entire bridge deck such that the evaporation rate is less than 0.2 lb/sq ft/hr. This may require placing the deck at night, in the early morning or on another day. The evaporation rate (as determined in the American Concrete Institute Manual of Concrete Practice 305R, Chapter 2) is a function of air temperature, concrete temperature, wind speed and humidity. When the evaporation rate is equal to or above 0.2 lb/ft2/hr, take actions (such as cooling the concrete, installing wind breaks, sun screens etc.) to create and maintain an evaporation rate less than 0.2 lb/ft2/hr on the entire bridge deck.

Respondent information:
State: <u>KY</u>   Name: <u>JOHNNY SPAULIDNG</u>   Position: <u>ENGINEERING TECH</u>
Follow up contact info:
phone number: <u>502-564-3160</u>   email address: <u>johnny.spaulding@ky.gov</u>

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? <u>90 f</u>
For structures or mass concrete? <u>90 f, mass 70 f</u>
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   X No
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? ☐Yes   ☐ No
If yes:
What type and severity of deterioration? _____
What is the expected reason that the concrete was hot?
          ☐High ambient temperature
          ☐High concrete temperature upon arrival
          ☐Other
          If other, please explain: _____

          Where is the damage located?
          ☐Pavement   ☐Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) <u>Chillers, Ice, Water Aggregate Stockpiles</u>
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☐Yes X No
If yes, please explain: _____

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? Contractor

Any additional comments regarding hot weather concrete placement? <u>I personally believe 90 f should be the maximum.</u>

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Respondent information:
State: Maine   Name: Michael Redmond   Position: Concrete Mix Designer
Follow up contact info:
phone number: 207-624-3271   email address: michael.redmond@maine.gov

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? 85
For structures or mass concrete? 85
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   ☒No
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? ☐Yes   ☒
No
If yes:
What type and severity of deterioration? _____
What is the expected reason that the concrete was hot?
        ☐High ambient temperature
        ☐High concrete temperature upon arrival
        ☐Other
        If other, please explain: _____

        Where is the damage located?
        ☐Pavement   ☐Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) Ice in the Mix, spraying aggregate stockpiles, windbreaks, early morning or night placements.
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used. ☒Yes ☐No
If yes, please explain: Maine DOT specifies an evaporation rate limit based on ambient temperature, mix temperature, wind speed, and relative humidity. Contractor is required to provide all equipment.
 All mixes under 70 degrees discharge within 90 minutes from the time cement is added to the mix. Mixes between 70 and 80 degrees discharge in 60 minutes .If a retarder is used discharge is 90 minutes. All mix over 80 degrees is 60 minutes with an 85 degree max placement temperature.

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? Contractor bears full responsibility for all costs.

Any additional comments regarding hot weather concrete placement? Maine does not experience prolonged periods of hot weather so this issue only happens a few weeks per year typically.

Respondent information:

State: <u>MI</u>   Name: <u>John Belcher</u>   Position: <u>Concrete Engineer</u>

Follow up contact info:

phone number: <u>517-322-5695</u>   email address: <u>belcherj@michigan.gov</u>

<u>Survey questions:</u>

In your state, what is the maximum allowable temperature for concrete placement:

For pavements? <u>The max temp of the concrete is 90F.</u>

For structures or mass concrete? <u>The max temp of the concrete is 90F for structures. Our Mass Pour spec is currently in the review process.</u>

In your state, are there any local variations in policy (i.e. by district)? ☐Yes   ☒No

If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? ☐Yes   ☒No

If yes:

What type and severity of deterioration? _____

What is the expected reason that the concrete was hot?

☐High ambient temperature

☐High concrete temperature upon arrival

☐Other

If other, please explain: _____

Where is the damage located?

☐Pavement   ☐Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) <u>We use night paving and night pours to combat hot weather.</u>

In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used. ☐Yes ☒No

If yes, please explain: _____

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? <u>Contractor</u>

Any additional comments regarding hot weather concrete placement? _____

Respondent information:

State: <u>MN</u>   Name: <u>Maria Masten</u>   Position: <u>Concrete Engineer</u>

Follow up contact info:

phone number: <u>651-366-5572</u>   email address: <u>maria.masten@dot.state.mn.us</u>

Survey questions:

In your state, what is the maximum allowable temperature for concrete placement:

For pavements? <u>Concrete temp of 90 degrees F but we usually allow up to 92F</u>

For structures or mass concrete? <u>Same spec as above for structures. For mass concrete we require thermal models, and curing temperatures that have a maximum concrete temperature of 160F. There is also a 45F temperature differential between the geometric center and 2" from an exposed face.</u>

In your state, are there any local variations in policy (i.e. by district)? ☐Yes   ☒No

If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? ☐Yes   ☒No

If yes:

What type and severity of deterioration? _____

What is the expected reason that the concrete was hot?

☐High ambient temperature

☐High concrete temperature upon arrival

☐Other

If other, please explain: _____

Where is the damage located?

☐Pavement   ☐Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) <u>sprinkling the aggregate piles, using cold water, sometimes allowing admixtures</u>

In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used. ☐Yes ☒No

If yes, please explain: _____

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? <u>Contractor</u>

Any additional comments regarding hot weather concrete placement? _____

Respondent information:
State: <u>MO</u>   Name: <u>Brett Trautman</u>   Position: <u>Field Materials Engineer</u>
Follow up contact info:
phone number: <u>(573)751-2926</u>   email address: <u>brett.trautman@modot.mo.gov</u>

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? <u>No specification limit</u>
For structures or mass concrete? <u>Max. 85 deg F (concrete for bridge decks)</u>
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   X No
If yes, please explain: <u>   </u>

Is there any deterioration in concrete attributed to high plastic temperatures? ☐Yes   X No
If yes:
What type and severity of deterioration? <u>Low to moderate cracking has been observed</u>
What is the expected reason that the concrete was hot?
         X High ambient temperature
         ☐High concrete temperature upon arrival
         X Other
         If other, please explain: <u>   </u>

         Where is the damage located?
         ☐Pavement   X Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) <u>MoDOT allows the use of ice chips, spraying aggregate stockpiles and liquid nitrogen. Contractor may use other methods if approved by the engineer and does not affect the quality of the concrete. Ice chips and spraying aggregate stockpiles are the common practices used. Some ready mix producers in the urban areas have water chiller systems. In the summer it is not uncommon for bridge deck placement to begin in the early morning (i.e. 4:00 a.m.) or the early evening (8:00 p.m.) in order to avoid the high ambient air temperatures.</u>

In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☐Yes X No
If yes, please explain: <u>   </u>

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? <u>MoDOT specifies the maximum concrete temperature and lets the contractor select the best method for complying with the specified requirement. The contractor would be responsible for this cost. The cost of cooling the concrete should be contained in the contractors bid price.</u>

Any additional comments regarding hot weather concrete placement? <u>   </u>

Respondent information:
State: **MS**   Name: **Adam Browne**   Position: **Concrete Field Engineer**
Follow up contact info:
phone number: **601-359-1761**   email address: **abrowne@mdot.state.ms.us**

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? **NONE**
For **structures** or mass concrete (**we don't typical make considerations for mass concrete**)?

> 1.   **90°F (32.2°C). for concrete mixture with less than 20% replacement of cement by fly ash or 45% replacement of cement by weight;**
> 2.   **95°F (35°C) for concrete mixture for drilled shafts;**
> 3.   **95°F (35°C) for concrete mixture more than 20% replacement of cement by fly ash or 45% replacement of cement by weigh**

In your state, are there any local variations in policy (i.e. by district)? ☐Yes   ☒**No**
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? ☐Yes   ☒
None in Mississippi of which I am aware
If yes:
What type and severity of deterioration? _____
What is the expected reason that the concrete was hot?
☐High ambient temperature
☐High concrete temperature upon arrival
☐Other
If other, please explain: _____

Where is the damage located?
☐Pavement   ☐Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) **Ice, Spraying aggregate stockpiles**
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☐Yes ☒**No**
If yes, please explain: _____

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? **Contractor receives no additional compensation**

Any additional comments regarding hot weather concrete placement? _____

Respondent information:
State: <u>NC</u>   Name: <u>Randy Pace</u>   Position: <u>State Materials Operations Eng</u>
Follow up contact info:
phone number: <u>9197337091</u>   email address: <u>rpace@dot.state.nc.us</u>

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? <u>90 F Air, 95 F concrete</u>
For structures or mass concrete? <u>95 F Concrete</u>
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   xNo
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? xYes   ☐ No
If yes:
What type and severity of deterioration? <u>Sometimes the curing is not adequate and we get shrinkage cracks but this is very rare.</u>
What is the expected reason that the concrete was hot?
   X High ambient temperature
   ☒High concrete temperature upon arrival
   ☒Other
   If other, please explain: <u>When it does occur it is normally a combination of high ambient temps , high concrete temps and high wind speeds and usually in the middle to end of summer.</u>

   Where is the damage located?
   ☐Pavement   x Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) <u>Cooling the mix water with ice, and saturating the aggregate stockpiles.</u>
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☐Yes x No
If yes, please explain: _____

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? <u>Contractor</u>

Any additional comments regarding hot weather concrete placement? _____

Respondent information:

State: <u>NE</u>   Name: <u>Wallace Heyen</u>   Position: <u>Portland Cement Concrete Engineer</u>

Follow up contact info:

phone number: <u>402-479-4677</u>   email address: <u>wheyen@dor.state.ne.us</u>

Survey questions:

In your state, what is the maximum allowable temperature for concrete placement:

For pavements?

<u>Temperature requirement – No. We have rate of evaporation requirements.</u>

For structures or mass concrete?

<u>Structures – No.</u>

<u>Bridge decks – 90 degrees (see table below)</u>

In your state, are there any local variations in policy (i.e. by district)? ☐Yes   ☒No

If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? ☒Yes   ☐ No

If yes:

What type and severity of deterioration? <u>Limited-shrinkage cracks</u>

What is the expected reason that the concrete was hot?

☒High ambient temperature

☐High concrete temperature upon arrival

☐Other

If other, please explain: _____

Where is the damage located?

☐Pavement   ☒Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) <u>No.</u>

In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used. ☒Yes ☐No

If yes, please explain: _____

For pavement the following specification applies:

k.   If the rate of evaporation approaches .2 lb/SF/h (1.0 kg/m^2/h), the Contractor must notify the Engineer regarding the additional actions that will be taken to prevent plastic shrinkage cracking. The rate of evaporation shall be obtained by using the nomograph at Figure 710.01.

<u>(Continued on next page)</u>

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Figure 710.01

EVAPORATION NOMOGRAPH
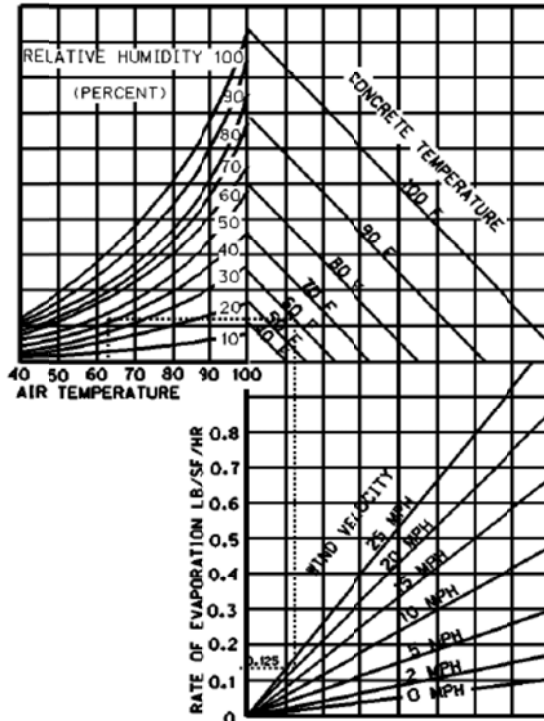
TO USE THIS CHART:
1. Enter with air temperature, move up to reative humidity.
2. Move right to concrete temperature.
3. Move down to wind velocity.
4. Move left: read approx. rate of evaporation.

IF THE EVAPORATION RATE IS 0.15 lb/sf/h, THEN THE CONTRACTOR SHALL TAKE
ACTIONS TO PREVENT PLASTIC SHRINKAGE CRACKING.

For bridge decks the following specification applies:

Bridge floor concrete shall not be placed when the anticipated wind velocity during the concrete placement period is expected to exceed the limitations shown in Table 706.01

Table 706.01

Temperature and Wind Velocity Limitations	
Air Temperature in the Shade, (Degrees Fahrenheit)	Maximum Wind Velocity, (Miles Per Hour)
86	9
77	16
68	22
59	28
50	40

Bridge floor concrete shall not be place when the ambient air or concrete temperature is above 90 degrees Fahrenheit.

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? No.

Any additional comments regarding hot weather concrete placement? No.

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Respondent information:
State: <u>NH</u>   Name: <u>Denis Boisvert</u>   Position: <u>Chief of Materials Technology</u>
Follow up contact info: phone number: <u>603-271-3151</u>
email address: <u>dboisvert@dot.state.nh.us</u>

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? <u>N/A</u>
For structures or mass concrete? <u>None</u>
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   ☒No
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? ☐Yes   ☒
No
If yes:
What type and severity of deterioration? _____
What is the expected reason that the concrete was hot?
          ☐High ambient temperature
          ☐High concrete temperature upon arrival
          ☐Other
          If other, please explain:

          Where is the damage located?
          ☐Pavement   ☐Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) <u>Addition of water-reducing or retarding admixtures; nighttime or morning placements</u>
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☐Yes ☒No
If yes, please explain: _____

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? <u>Contractor</u>

Any additional comments regarding hot weather concrete placement? Here is our spec, we do not have a maximum ambient temp but we do have evaporation limits using temp.
3.8.1 General.
3.8.1.1 The Contractor shall maintain the gas delivered concrete below 85°F (30°C). This may require the addition of ice to the mixing water, sprinkling the forms and reinforcing steel, scheduling the concrete placements for early morning or evening hours, or any other approved methods.
3.8.1.2 After placement, the concrete shall be maintained below 100°Fby sunshades, windscreen, fog spray, wet burlap, or other approved methods.
3.8.1.3 If excess water demand or too rapid setting is anticipated, a Water-Reducing or Retarding Admixture may be added to the mix in the amounts ordered.
3.8.1.4 When daytime air temperatures exceed 85 °F (30 °C), concrete temperature exceeds 80°F (26.6°C)(25°C), and in the opinion of the Engineer, satisfactory surface finish

cannot be achieved, nighttime or early morning placement may be required. No additional compensation will be allowed for this work.

Respondent information:
State: **NEW JERSEY** Name: **Donald S Matlack**   Position: **Reg'l Materials Engr**
Follow up contact info:
phone number: **(856) 486-6612**   email address: **donald.matlack@dot.state.nj,us**

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? **90°F for plastic conc, 85°F ambient**
For structures or mass concrete? **90°F for plastic conc; 85°F ambient is a hard maximum for bridge decks. For bridge decks & approaches, evaporation rate must be less than 0.15#/sf-hr unless fog misting is used.**
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   ☒**No**
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? ☐Yes   ☒
No
If yes:
What type and severity of deterioration? _____
What is the expected reason that the concrete was hot?
        ☐High ambient temperature
        ☐High concrete temperature upon arrival
        ☐Other
        If other, please explain: _____

        Where is the damage located?
        ☐Pavement   ☐Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) **All ready-mix producers sprinkle aggregate stockpiles. Some are equipped with water chillers, others will substitute ice for mixing water. We do not specify how they must meet the mix temperature requirement.**
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☐Yes ☒**No**
If yes, please explain**: "Mass" concrete and High Performance Concrete (HPC) must contain slag or fly ash, but this is not weather-dependent. Mass concrete requires thermal monitoring.**

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? **In general the contractor must include these costs in the bid price for the various concrete items.**

Any additional comments regarding hot weather concrete placement? _____

State: <u>NV</u>   Name: <u>Dean Weitzel</u>   Position: <u>Chief Materials Engineer</u>
Follow up contact info: phone number: <u>(775) 888-7520</u>
email address: <u>   </u>

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? <u>90°F</u>
For structures or mass concrete? <u>Must comply with the thermal control plan.</u>
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   X No
If yes, please explain: <u>   </u>

Is there any deterioration in concrete attributed to high plastic temperatures? ☐Yes   X No
If yes:
What type and severity of deterioration? <u>   </u>
What is the expected reason that the concrete was hot?
        ☐High ambient temperature
        ☐High concrete temperature upon arrival
        ☐Other
        If other, please explain: <u>   </u>

        Where is the damage located?
        ☐Pavement   ☐Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) <u>Yes</u>
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used. ☐Yes XNo
If yes, please explain: <u>   </u>

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? <u>Contractor</u>

Any additional comments regarding hot weather concrete placement? <u>No</u>

Respondent information:

State: <u>NY</u>　　　　　Name: <u>Donald Streeter</u>　　　　　Position: <u>Concrete Program Mgr</u>

Follow up contact info:

phone number: <u>518-457-4593</u>　　　email address: <u>dstreeter@dot.state.ny.us</u>

Survey questions:

In your state, what is the maximum allowable temperature for concrete placement:

For pavements?　<u>90F</u>

For structures or mass concrete?　<u>90F</u>

In your state, are there any local variations in policy (i.e. by district)? ☐Yes　　　☒No

If yes, please explain: <u>　　　</u>

Is there any deterioration in concrete attributed to high plastic temperatures? ☐Yes　　☐ No

If yes:

What type and severity of deterioration? <u>There is no certainty to directly tie longer term</u>
<u>deterioration to the higher placement temperature. We usually have 2 – 3 weeks in the</u>
<u>summer when we have extremely high temperatures and concrete is delivered at or near</u>
<u>90F (This is usually tied to very high cement temps). The result is that we see problems</u>
<u>achieving 28 day compressive strengths – the concrete sets quick, gains strength fast, and</u>
<u>any long term strength gain ceases very early – this could relate to winter surface scaling</u>
<u>problems we're seeing on flatwork(?)</u>

What is the expected reason that the concrete was hot?

　　　☒High ambient temperature

　　　☒High concrete temperature upon arrival

　　　☒Other

　　　If other, please explain: <u>high cement production / delivery temps.</u>

　　　Where is the damage located?

　　　☐Pavement　☐Bridge decks　　☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) <u>Specifications require that we don't place</u>
<u>concrete at high temps and the contractor / producer are responsible to provide concrete at</u>
<u>the project site that meets specification requirements – including max temp of 90F</u>

In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used. ☐Yes ☒No

If yes, please explain: <u>　　　</u>

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? <u>Contractor / producer are responsible to</u>
<u>deliver concrete meeting spec requirements.</u>

Any additional comments regarding hot weather concrete placement? <u>　　　</u>

Respondent information:
State: Ohio Department of Transportation   Name: Lloyd   Position:
Administrator Office of materials Management
Follow up contact info:
phone number: 6142751351   email address: Lloyd.welker@dot.state.oh.us

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? 90
For structures or mass concrete? 85 (mass concrete is separate controls)
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   ☒No
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? ☒Yes   ☐No
If yes:
What type and severity of deterioration? See cracking –some curling
What is the expected reason that the concrete was hot?
        ☒High ambient temperature
        ☒High concrete temperature upon arrival
        ☐Other
        If other, please explain: _____

        Where is the damage located?
        ☒Pavement   ☒Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) Don't
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used. ☐Yes ☒No
If yes, please explain: _____

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? No compensation. It's their schedule!

Any additional comments regarding hot weather concrete placement? _____

Respondent information:
State: <u>OK</u>   Name: <u>Kenny Seward</u>   Position: <u>Structural Materials Engineer</u>
Follow up contact info: phone number: <u>(405) 321-5198</u>   email address:
<u>kseward@odot.org</u>

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? <u>90</u>
For structures or mass concrete? <u>85</u>
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   ☒No
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures?
        ☐Yes   ☒No
If yes:
What type and severity of deterioration? _____
What is the expected reason that the concrete was hot?
        ☐High ambient temperature
        ☐High concrete temperature upon arrival
        ☐Other
        If other, please explain: _____

        Where is the damage located?
        ☐Pavement   ☐Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) <u>Ice or liquid nitrogen</u>
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☐Yes ☒No
If yes, please explain: _____

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? <u>There is no increased compensation. It is just something the contractor has to do.</u>

Any additional comments regarding hot weather concrete placement? <u>We have had concrete temperature requirements for so long it is not really a problem.</u>

Respondent information:
State: <u>Pennsylvania</u>   Name: Patricia Miller__ Position: <u>Unit manager</u>
Follow up contact info: phone number: 717-787-2489   email address: <u>pimiller@state.pa.us</u>

<u>Survey questions:</u>
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? <u>90° F</u>
For structures or mass concrete? <u>80° F</u>
In your state, are there any local variations in policy (i.e. by district)? ☒Yes
If yes, please explain: <u>N/A</u>

Is there any deterioration in concrete attributed to high plastic temperatures? ☒No
If yes:
What type and severity of deterioration? <u>N/A</u>
What is the expected reason that the concrete was hot?
      ☐High ambient temperature
      ☐High concrete temperature upon arrival
      ☐Other
      If other, please explain: _____

      Where is the damage located?
☐Pavement   ☐Bridge decks☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) <u>Cooling aggregate and mixing water and/or adding ice to the concrete</u>
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.   ☒No
If yes, please explain: <u>N/A</u>

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? <u>The cost is in the contractor's bid.</u>

Any additional comments regarding hot weather concrete placement? No

Respondent information:
State: <u>SC</u>   Name: <u>Ally Hussein</u>   Position: <u>Structural Materials Engineer</u>
Follow up contact info: phone number: <u>1-803-737-6687</u>   email address:
<u>husseinaa@scdot.org</u>

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements or structures?   <u>90° F</u>
For mass concrete?   <u>80° F</u>
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   X No
If yes, please explain: <u>   </u>

Is there any deterioration in concrete attributed to high plastic temperatures? ☐Yes   X No
If yes:
What type and severity of deterioration? <u>   </u>
What is the expected reason that the concrete was hot?
        ☐High ambient temperature
        ☐High concrete temperature upon arrival
        ☐Other
        If other, please explain: <u>   </u>

        Where is the damage located?
        ☐Pavement   ☐Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g.
ice chips, spraying, ponding, liquid nitrogen, etc.) <u>ice chips and sprinkling coarse aggregate</u>
<u>with water</u>
In your state, are there any weather-related restrictions on mix design? For example, if
concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement
ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☐Yes X No
If yes, please explain: <u>   </u>

How is compensation to contractors handled? For example, who absorbs the costs for
cooling concrete or lost time due to hot weather? <u>No compensation</u>

Any additional comments regarding hot weather concrete placement? <u>Before starting work</u>
<u>on any project, the contractor must submit a written "**Hot Weather Batching and Mixing**</u>
<u>**Plan"** developed in conjunction with the concrete supplier .</u>

State: SD   Name: Darin Hodges   Position: Concrete Engineer
Follow up contact info: phone number: 605-773-7193
email address: Darin.Hodges@state.sd.us

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? 90
For structures or mass concrete? 90
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   XNo
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? XYes   ☐
No
If yes:
What type and severity of deterioration? Additional Cracking
What is the expected reason that the concrete was hot?
        x☐High ambient temperature
        x☐High concrete temperature upon arrival
        x☐Other
        If other, please explain: Lack of additional Curing due to Higher Temps.

        Where is the damage located?
        ☒Pavement   ☒Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g.
ice chips, spraying, ponding, liquid nitrogen, etc.) Watering Coarse aggregate piles, & Ice.
In your state, are there any weather-related restrictions on mix design? For example, if
concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement
ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
x☐Yes   ☐No
If yes, please explain: On Bridge Decks, the Max temp is 80 deg, but may be raised to 85 if
the following are done: Flush Coarse Agg. Piles with water for 24 Hours, Maintain a min.
rate of pour of 40 yd 3 per hour, Wet Burlap & Poly sheeting placed immediately after finish
machine, & Continuous fogging be applied.

How is compensation to contractors handled? For example, who absorbs the costs for
cooling concrete or lost time due to hot weather? Contractor Absorbs the costs to keep the
project moving.

Any additional comments regarding hot weather concrete placement? _____

I request a summary of the results.

Respondent information:
State: Texas   Name: Lisa Lukefahr   Position: Manager, Rigid Pavements and Concrete Materials
Follow up contact info: phone number: 512.506.5858   email address: elukefa@dot.state.tx.us

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? 95 deg F
For structures or mass concrete? 95 deg F for structures; for mass concrete a plan must be approved to keep the maximum core temperature less than 160 deg F
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   ☒No
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? ☒Yes   ☐ No
If yes:
What type and severity of deterioration? DEF – serious to moderate; mild to moderate pavement behavior issues because of zero-stress temperature (warping/curling/cracking/etc.); mild to moderate drying shrinkage cracking;
What is the expected reason that the concrete was hot?
          ☒High ambient temperature
          ☒High concrete temperature upon arrival
          ☒Other
          If other, please explain: High cement factor

          Where is the damage located?
          ☒Pavement   ☒Bridge decks   ☒Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) Liquid nitrogen is growing in popularity, especially at urban commercial ready mix plants, but use is still limited to a handful of suppliers; the use of ice, followed by chilled water, and supplemented with sprinkling stockpiles is the most common.
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used. ☒Yes ☐No
If yes, please explain: Many concrete pavement projects use a plan note requiring the use of Class F fly ash from April to October; when ambient temperatures are in excess of 85 deg F, a retarder is required in all superstructure concrete.

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? Subsidiary to pay item.

Any additional comments regarding hot weather concrete placement? The development of Concrete Works was specifically to aid in concrete temperature predictions, to avoid DEF, thermal gradients/shock in mass concrete, etc. Version 2 also includes pavement temperatures, time to corrosion, etc. It is located at www.TexasConcreteWorks.com.

Respondent information:
State: Virginia   Name: Larry J. Lundy   Position: Concrete Program Engineer
Follow up contact info:
phone number: 804-328-3130   email address:
Larry.Lundy@VDOT.Virginia.gov

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? 95 ^{0}F
For structures or mass concrete? 85 ^{0}F or as specified by the contract. Contracts with
mass concrete may list a maximum temperature differential from the most interior point to
the surface.
In your state, are there any local variations in policy (i.e. by district)? ☐Yes
☒None that have been reported.
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? ☐Yes
☒None that have been documented.
If yes:
What type and severity of deterioration? _____
What is the expected reason that the concrete was hot?
          ☐High ambient temperature
          ☐High concrete temperature upon arrival
          ☐Other
          If other, please explain: _____

          Where is the damage located?
          ☐Pavement   ☐Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g.
ice chips, spraying, ponding, liquid nitrogen, etc.) cooling the aggregate by spraying, cooling
the water using a chiller, adding ice to the concrete mix, spraying the sub grade and forms.
In your state, are there any weather-related restrictions on mix design? For example, if
concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement
ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☐Yes ☒No
If yes, please explain: _____

How is compensation to contractors handled? For example, who absorbs the costs for
cooling concrete or lost time due to hot weather? The Contractor.

Any additional comments regarding hot weather concrete placement? The Virginia DOT will
consider/review contractor proposals regarding the placing of concrete in hot weather.

Respondent information:
State: <u>Washington</u>   Name: <u>Jim Walter</u>   Position: <u>Asst St Mat Engr</u>
Follow up contact info:
phone number: <u>360-709-5410</u>   email address: <u>walteji@wsdot.wa.gov</u>

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? <u>90F</u>
For structures or mass concrete? <u>90F</u>
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   X☐No
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? ☐Yes   X
☐No
If yes:
What type and severity of deterioration? <u>None</u>
What is the expected reason that the concrete was hot?
            ☐High ambient temperature
            ☐High concrete temperature upon arrival
            ☐Other
            If other, please explain: _____

            Where is the damage located?
            ☐Pavement   ☐Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g.
ice chips, spraying, ponding, liquid nitrogen, etc.) _____
In your state, are there any weather-related restrictions on mix design? For example, if
concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement
ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☐Yes X☐No
If yes, please explain: _____

How is compensation to contractors handled? For example, who absorbs the costs for
cooling concrete or lost time due to hot weather? _____

Any additional comments regarding hot weather concrete placement? _____

Respondent information:
State: <u>WI</u>   Name: <u>James Parry</u>   Position: <u>Concrete Engineer</u>
Follow up contact info:
phone number: <u>608-246-7939</u>   email address: <u>james.parry@dot.state.wi.us</u>

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements? <u>No limit</u>
For structures or mass concrete? <u>80 F</u>
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   ☒No
If yes, please explain: _____

Is there any deterioration in concrete attributed to high plastic temperatures? ☒Yes   ☐ No
If yes:
What type and severity of deterioration? <u>Increased risk of early-age cracking</u>
What is the expected reason that the concrete was hot?
        ☒High ambient temperature
        ☒High concrete temperature upon arrival
        ☐Other
        If other, please explain: _____

        Where is the damage located?
        ☒Pavement   ☐Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g. ice chips, spraying, ponding, liquid nitrogen, etc.) <u>Ice is typically used for structure concrete</u>
In your state, are there any weather-related restrictions on mix design? For example, if concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☐Yes ☒No
If yes, please explain: _____

How is compensation to contractors handled? For example, who absorbs the costs for cooling concrete or lost time due to hot weather? <u>WisDOT pays for ice.</u>

Any additional comments regarding hot weather concrete placement? <u>We have good, tight controls for structure concrete, but need to improve our pavement specs for hot weather placement.</u>

Respondent information:
State: WV   Name: Mike Mance   Position: Cement & Concrete Unit Leader
Follow up contact info:
phone number: (304) 558-9846 email address:Mike.A.Mance@wv.gov

Survey questions:
In your state, what is the maximum allowable temperature for concrete placement:
For pavements?   90 degrees (plastic concrete temperature)
For structures or mass concrete?   90 degrees (plastic concrete temperature)
In your state, are there any local variations in policy (i.e. by district)? ☐Yes   X No
If yes, please explain:

Is there any deterioration in concrete attributed to high plastic temperatures? ☐Yes   X
No
If yes:
What type and severity of deterioration?
What is the expected reason that the concrete was hot?
      ☐High ambient temperature
      ☐High concrete temperature upon arrival
      ☐Other
      If other, please explain:

      Where is the damage located?
      ☐Pavement   ☐Bridge decks   ☐Secondary (e.g. barrier walls)

In your state, what is the standard practice for placing concrete in high temperatures? (e.g.
ice chips, spraying, ponding, liquid nitrogen, etc.)   Ice
In your state, are there any weather-related restrictions on mix design? For example, if
concrete could be placed at temperatures above 90°F (32.2°C). only if the water-cement
ratio is above a certain value, or certain mineral admixtures such as slag or fly ash are used.
☐Yes X No
If yes, please explain:

How is compensation to contractors handled? For example, who absorbs the costs for
cooling concrete or lost time due to hot weather?   Contractors absorb the cost. Ready mix
suppliers build any additional cost of cooling the mix into their price which is passed along to
the Contractor. Lost time due to hot weather is absorbed by the Contractor in the same way
as lost time due to cold weather or rain.

Any additional comments regarding hot weather concrete placement?

APPENDIX E: A REVIEW OF RING SHRINKAGE TESTS FOR CONCRETE

INTRODUCTION AND BACKGROUND

Volumetric changes (shrinkage or expansion) in Portland cement based concrete occur due to temperature and moisture changes or by chemical reactions. Three types of shrinkage can contribute to cracking in concrete: plastic, drying, and autogenous shrinkage. *Plastic shrinkage* occurs in cement paste in its plastic state. Rapid loss of moisture caused by exterior forces such as evaporation can lead to a volume reduction in the cement paste even before concrete has gained strength. This volumetric shrinkage when the rate of moisture removal at the surface exceeds the rate at which bleed water rises to the surface, causes cracks in the plastic concrete due to negative capillary pressures applied on the cement skeleton [Mindess, J; Darwin, D. et al [2003].

Drying shrinkage results from a loss of water from the concrete. It is the reduction in concrete volume resulting from a loss of water from the concrete and is directly related to the removal of adsorbed water held by hydrostatic tension in small capillaries (<50 nm) of the hydrated cement paste and its magnitude is highly dependent on the amount of water lost and on the rate of this evaporation.

Autogenous shrinkage of cement paste and concrete is defined as the macroscopic volume change occurring with no moisture transferred to the exterior surrounding environment. It is a result of chemical shrinkage affiliated with the hydration of cement particles. In concretes with water-cement ratios greater than 0.40 autogenous shrinkage is generally small and is commonly included as part of the drying shrinkage [Nevill, A.M. et al [1996].

OBJECTIVES

The purpose of this literature review is to look at tests that measure concrete shrinkage with particular attention in adapting a shrinkage test to assess the effects of high temperature concrete placement on the cracking potential of concrete, e.g., ring shrinkage test.

FACTORS INFLUENCING SHRINKAGE

Factors affecting the overall shrinkage in Portland cement based materials are: characteristics and quantities of mineral admixtures, the amount of water, and thus the water-cement ratio. The water-cement ratio controls the evaporable water content per unit volume of paste and the rate at which water can reach the surface. For mixes with the same water-cement ratio, shrinkage increases with increases in cement content because the volume of hydrated cement, or paste, also increases. Cement fineness can affect the drying shrinkage of concrete. Larger cement particles that do not undergo full hydration can provide a restraining effect similar to that of aggregates. For this reason, shrinkage values tend to be greater for finer cements [Mehta, P.K., et al [1994]. [Chariton; Weiss, W.J. et al [2002] stated that finer cements resulted in a finer pore structure, which caused higher capillary stresses and increased shrinkage. According to [Mehta, P.K., et al [2002] modern portland cements are ground to a fineness that is close to 400 m^2/kg Blaine, a value considerably higher than the typical fineness of older cements, namely 250-300 m^2/kg. This high fineness increases the reactivity of the cement and the liberation of heat of hydration, situation that could be aggravated especially in hot weathers in which the adiabatic

temperature rise in concrete mixtures can become a liability. According to the Committee 305 of the [American Concrete Institute (ACI) et al [1996], concrete produced under hot weather conditions presents increased shrinkage problems due to an increase in the water content of concrete caused by a higher demand of in the mixing water.

The volume fraction of aggregates in the concrete mixes also affects the amount of shrinkage that increases at a much greater rate with decreasing aggregate volumes [Odman, S; et al [1968].   [Neville, A.M. et al [1996] affirm that the aggregate restrains shrinkage of the cement paste; aggregates provide restraint because they do not undergo moisture volume changes. The amount, size, and elastic properties of an aggregate determine the amount of restraint it provides [Mindess, J.; Young F, Darwin D. et al [2003], [Neville, A.M. et al [1996]; [Pickett, G, et al [1956]; [Reichard, t.W. et al [1964]. Admixtures have effects on shrinkage varying from none to a substantial amount. Air-entraining agents have little effect if mix proportions are adjusted properly. Shrinkage also increases with the use of some admixtures such as superplasticizers or those admixtures that increase the amount of water in the mix [Ma et al. [2004]. Shrinkage-reducing admixtures have been proved to be effective to improve shrinkage resistance and cracking behavior when used in concrete mixes [Shah. S.P.; Weiss, W.J.; Yang, W. et at [1998]; [See, H.T., Attiogbe, E. K.; Mittenberger, M.a. et al [2003]. These admixtures reduces surface tension of the mix water, which in turn reduces the stresses in the capillary pores [Shah, S.P.; Weiss, W. J., Yang, W.[1998]; [Shah, S. P.; Karogular, M.E.; Sarigaphutt, M. et al [1992].

FREE SHRINKAGE MEASUREMENTS

Free Shrinkage Tests

Free shrinkage tests help to determine the potential of material to shrink. Existing free shrinkage tests are typically simple and mainly involve measurement of reduction in a certain dimension of the specimens [Kovier, K. et al [1994]. The specimens are usually in the shape of a prism, the most common procedure to test free shrinkage is described in ASTM C 157, "Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete." In this test method, rectangular concrete prisms are cast with gage studs at either end. A length comparator is used to measure shrinkage relative to an initial reading. However, a free shrinkage test alone is not enough to determine whether cracking can be expected in a structure because time dependent material properties such as creep, stiffness and toughness also influence the potential for cracking [Moon, J.; Rajabipour, F, Pease B & Weiss, J. et al. 2006]; [Hossain, A. B., Pease, B., Weiss, J. et al. 1834]. Consequently, it is necessary to perform restrained shrinkage tests to assess the effects of shrinkage on stress development in the concrete specimen.

Restrained Shrinkage Measurements

When an element is prevented by external restraint to contract due to fixed ends, reinforcing bars, sub-base friction, inclusions, girder systems, etc, tensile stresses develop that may be sufficient to cause cracking if they exceed the tensile strength of the concrete. Shrinkage cracking due to restraint depends on the free shrinkage and is also influenced by time dependent development of material parameters including: stiffness, fracture resistant, creep relaxation, and the degree of restraint. Several Researchers [Moon, J., Rajabipour, F, Pease B. & Weiss, J. et al [2006] have used linear specimens restrained at both ends to investigating shrinkage cracking. The use of these linear geometries results in the development of uni-axial tensile stresses which can easily be related to specific structural applications [Weiss, J., Yang, W. et al [1997]. The restrained ring test has been used by numerous researchers to assess the potential for shrinkage cracking in concrete mixtures [Weiss, J., Yang, W. et al [1997]. Many researchers have evaluated the shrinkage and

cracking behavior of concrete using a variety of test procedures and specimens. The most common restrained shrinkage tests have used flat, plate-type specimens, long and thin linear specimens, or ring specimens. In many of the studies, free shrinkage tests have accompanied the restrained shrinkage test.

Plate Tests

In order to study cracking caused by plastic shrinkage, [Kraai, P., et al (1985] developed a procedure to evaluate the effect of fibers in reducing shrinkage cracking in concrete slabs. In this test, two small thin concrete slabs (one for control and one for rapid drying test) were cast side by side and exposed to identical severe drying conditions for 24 hours, thereby increasing the cracking tendency of the concrete. To approximate the geometric proportions of a typical concrete slab, the thick of the plate specimens was 19 millimeter (¾ in.) wood forms with the bottom lined with plastic were used to prohibit absorption and reduce restraint. Evaporation and shrinkage rates were accelerated by the small thickness and large surface area. Relative cracking potential was determined by comparing the test panel with the control panel. [Kraai, P, et al (1985] found that cracking began around one hour after drying was initiated and most of the cracking occurred within 4 hours.

[Shaeles, C., Hover, K.C., et al [1980] used a similar test procedure to [Kraai P, et al [1985] to evaluate how mix proportions and construction practices affect plastic shrinkage in concrete. To improve durability and prevent absorption, the authors used plexiglass forms with the same thickness used by [Kraai, Pm et al [1985] (see figure 1). The slabs were cast using mortar only because the use of coarse aggregates in the mix would introduce unrealistically disproportionate restraint and discontinuity. Temperature, relative humidity and wind speed were controlled to simulate hot weather concreting conditions. The authors observed that cracking initiated earlier and stopped quicker in stiffer mixes and when the air temperature was higher. Crack widths and total crack areas were lower for stiff mixes compared to fluid and semi-plastic mixes.
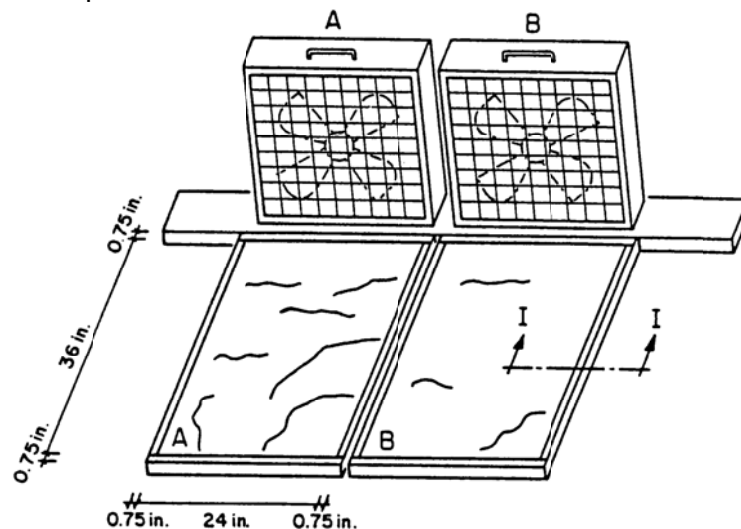
Figure 23. General arrangement of the test used by Kraai [1985] and Shaeles and Hover [1988].

To demonstrate a plate-type test procedure for quantifying the effect of the fiber addition on shrinkage and crack control, [Padron, I.; Zollo, R. F. et al [1990] studied the effects of adding low modulus synthetic fibers to concrete and mortar mixes. Figure 24 shows the test section of approximately 0.09 m (1 ft) square plate specimens with either 12

or 25 mm (½ or 1 in) thickness for mixes with small aggregates or mixes with large aggregates respectively. A steel ring in the center of the specimen was used to provide restraint against shrinkage. After casting, the concrete samples were placed in a wind tunnel and subjected to a drying environment of 31 °C (88 °F) and 50% relative humidity for 16 hours. The specimens were kept in the molds so that only the top surface was exposed. After 16 hours of drying, the top surfaces were polished with a series of coarse to fine sandpapers so that the cracks could be seen more readily. These cracks were then measured for length and width to calculate overall shrinkage and crack area. The authors observed that cracks began to form in the mortar samples 1¾ to 2 hours after drying started. Cracks initiated in the concrete specimens 1½ to 2 hours after drying began, and most cracking occurred within the first 6 hours for both types of mixes.
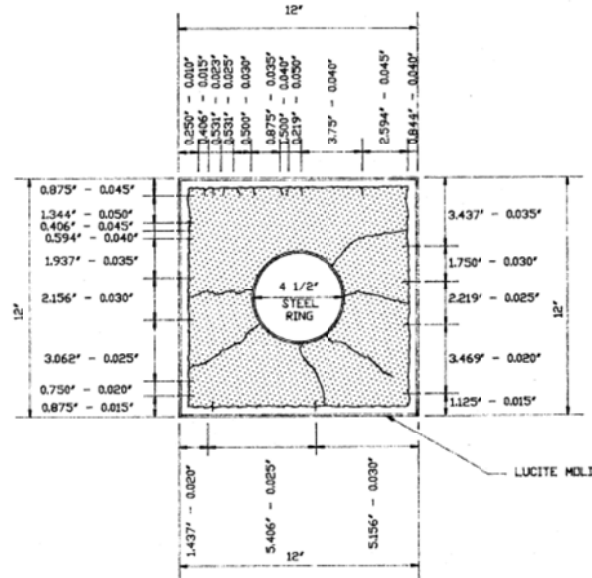
Figure 24. Typical specimen for measurement of overall shrinkage used by Padron and Zollo [1990].

Linear Tests

The risk of early age shrinkage cracking can be assessed by tests of restrained specimens. Several methods reported in the literature such as the ring specimen test [Weiss, J., Yang, W., et al [1997] and the uni-axial specimen test [Paillere, A.M., Buil, M, Serrano, J.J., et al [1989] provide results limited to the measurement of stresses and the potential for cracking and could not be accurate in hot weather environments. A restrained uni-axial test was developed to overcome limitations of previous experiments, allowing the measurement of tensile creep in a restrained test [Bloom, R., Bentur, A., et al [1995]; [Attoubat, S.A., et al [2000]. This restrained uniaxial test reveals the real contribution of tensile creep in relaxing shrinkage stresses, and provides valuable information for theoretical modeling.

[Paillere, A.M., Buil, M, Serrano, J.J., et al [1989] studied the restrained autogenous shrinkage behavior of concrete with steel fibers. They performed a self-cracking test on concrete using a cracking-test bench as shown in figure 25. With this apparatus, a concrete specimen is cast in a mold on a horizontal bench. The specimen is placed in a vertical position after the concrete sets to prevent any bending effects. The specimen has a cross section of 8.5 x 12 cm (3.4 x 4.7 in.) and a total length of 1.50 m (59 in.). The ends of the specimen are enlarged to fit into grips on the testing apparatus. One end is fixed, while the other is mobile to allow for shrinkage. A monitoring system at the mobile end controls a

dynamometer that applies and records the force required to keep the specimen at a constant length. The restrained shrinkage stress is calculated from this force and the cross-sectional area. To measure and compare the free shrinkage behavior of the concrete, a companion specimen with the same geometry is cast in a mold that allows it to shrink freely at one end. Two different sizes of steel fibers were used, one size at a time, in three of the mixes. The addition of steel fibers to concrete was found to increase the time to cracking and restrict crack width development.
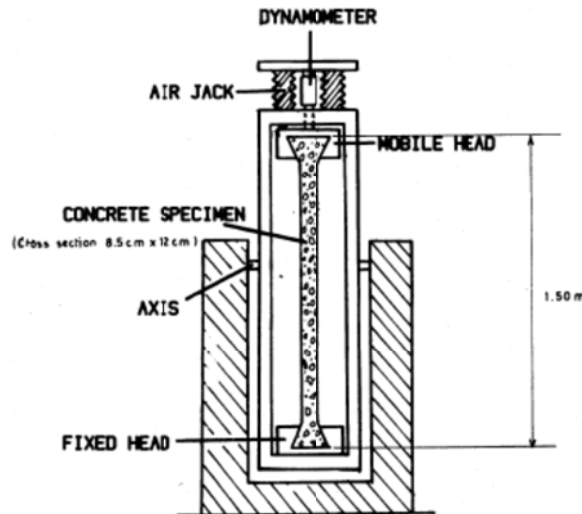
Figure 25. Self-cracking test apparatus used by Paillere et al. [1989].

[Bloom, R., Bentur, A. et al. [1995] modeled their restrained shrinkage test after the one developed by [Paillere, A.M., Buil, M., Serrano, J. J. et al. [1989]. In their test, the authors reduced the cross section of the concrete specimen to 40 x 40 mm (1.6 x 1.6 in.) and the length to 1000 mm (39.4 in.) as showed in figure 26. The ends of the specimen were enlarged and held in grips; one was free to move and the other was fixed. The mobile grip was connected to a screw assembly that was used to manually return the specimen to its initial length whenever its shrinkage reached 2 µm. Therefore, full restraint was maintained in a step-wise manner. The load applied in each step was determined through the use of a load cell that was connected inline with the screw assembly. The restrained shrinkage stress was then calculated from these loads. This test was designed so that the specimens were cast horizontally and testing could begin immediately after casting. The forms could be removed from the sides so the specimen can be dried from any combination of three sides. Three of the six mixes tested contained 15% silica fume. Mixes with low water/binder(w/b) ratios, both with and without silica fume, exhibited plastic shrinkage cracking, while one with no silica fume and a water/binder(w/b) of 0.50 did not. Plastic shrinkage cracking occurred in the 0.40 w/b mix with silica fume, but not in a similar mix without silica fume. The 0.40 w/b mix without silica fume exhibited cracking in the hardened concrete after 36 hours. Plastic shrinkage in unrestrained specimens was significantly increased by the addition of silica fume.
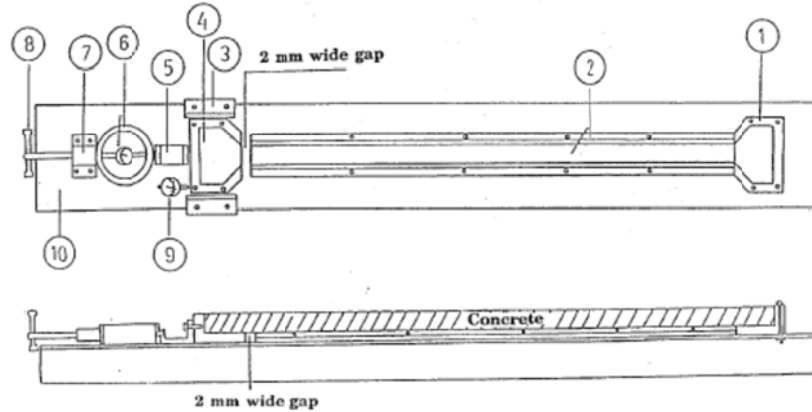
Figure 26. Restrained Shrinkage System used by Bloom et al. [1995]

[Kovler, K, Bentur, A., et al [1997] studied shrinkage in steel fiber reinforced concrete at early ages under hot climate conditions. They used a closed-loop, computer controlled uniaxial-restrained shrinkage testing device shown in Figure 27. This automated testing apparatus was used to measure strain components, and determine shrinkage induced stresses, modulus of elasticity, and tensile strength of the concrete. The effect of creep was determined by simultaneously evaluating twin specimens, one free and one restrained. The creep strain was the difference between the free shrinkage and restrained shrinkage strains. The concrete specimens were 40 x 40 mm (1.6 x 1.6 in.) in cross section and had a gage length of 1000 mm (39.4 in.). After one day of curing, the specimens were placed in a 32 ± 1 °C (89.6 ±1.8 °F), 35 ± 2% relative humidity drying environment. The major conclusions from this test were that the steel fiber reinforced concrete maintained load carrying capacity after initial cracking and that the cracks were distributed along the length of the specimens. Crack widths were also found to be small in this type of concrete.
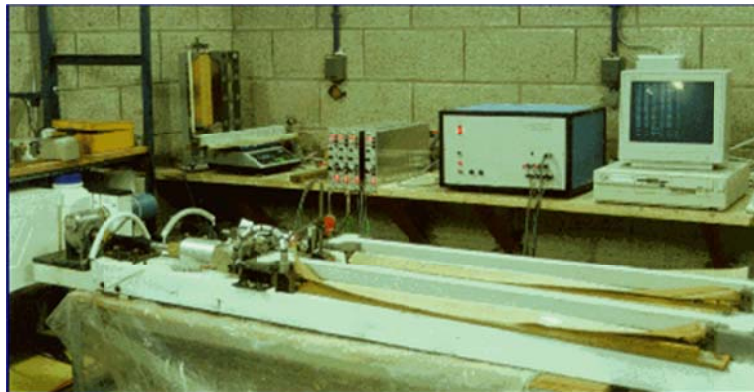
Figure 27. Shrinkage and creep of concrete in hot climate by Kovler and Bentur [1997].

[Jensen, O., & Hansen, P., et al [1995] developed a dilatometer called Auto-Shrink, designed especially for measuring autogenous deformation in cement paste combining the advantages of linear and volumetric measurements. The purpose of the Auto-Shrink system is to measure the unrestrained autogenous shrinkage of a specimen of cement paste or mortar cured under sealed conditions. Auto-Shrink permits evaluation of the relative autogenous shrinkage potential of different cementitious systems.

The dilatometer consists of a measuring frame and special moulds for encapsulation of the cement paste as shown in Figure 28. During the tests the dilatometer is submerged in a thermostatically controlled glycol bath. According to the authors, the measuring technique permits: (i) commencement of the measurements shortly after casting (1 hour after water addition in the present case); (ii) insignificant restraint of the hardening cement paste; (iii) accurate control of the temperature of the dilatometer and of the hardening cement paste; and (iv) efficient sealing of the cement paste. With Auto-Shrink it is possible to measure the time dependent deformation of many different specimens' simultaneously over weeks or even years (see Figure 29). Auto-Shrink is intended primarily for measurements after setting of cement pastes or mortars with a maximum aggregate size of 2 mm.

The Auto-Shrink digital dilatometer is composed to the following basic elements:
• A corrugated plastic mold with tight-fitting plugs to prepare a slender test specimen
• A rigid frame to support the specimen
• A digital dial gauge with remote control to measure change in specimen length
• A reference bar

Figure 28. Auto-Shrink digital dilatometer by Jensen and Hansen [1995].

Figure 29. Auto-Shrink digital dilatometer for two specimens [1995].

The autogenous shrinkage determination was carried out on a test rig composed by a corrugated PVC mold and immersed inside a temperature controlled bath as shown in

Figure 30, [email address – Multi-phipics]. External transducer measures the displacements of the upper head of the specimen while the cylindrical specimen is placed vertically designed. A thermocouple embedded in the core of the specimen measures the concrete temperature variations. The test rig designed at LCPC contains a thermostatically controlled water bath, a conditioning box and a PC in addition to the rig and the container. The measurements can start no more than one hour after the concrete batching (see Figure 31).

Figure 30. Test rig for monitoring Autogenous shrinkage in concretes.

The autogenous deformations begin at roughly equal to the setting time. Typically, the usual test is carried out in isothermal conditions (20°C), without moisture exchange with outside (the upper part of the mould is sealed by a cap) and at the atmospheric pressure (when the internal pressure differs from the atmospheric pressure of a value higher than 0.05 bar, the pressure equilibrium is provided by a valve fixed to the cap). For specific tests, the controlled temperature can follow a realistic history. This test rig measures the displacement of the upper head of the specimen but the measurement must be initialized since time to, a specific time determined by a parallel experimental method. The temperature changes used in this method are related to chemical reactions more than to mechanical processes and are not directly linked to the setting [Tanabe, t., et al [2009].
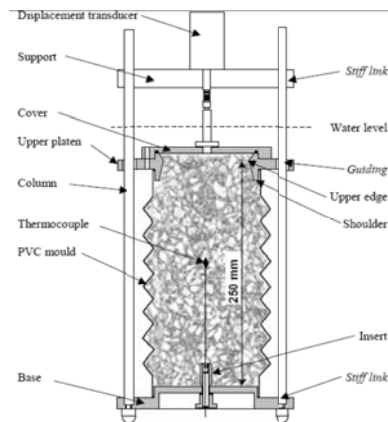
Figure 31. Test rig for concrete [2009].

Restrained tests have been used to evaluate the risk of early age cracking and the cracking sensitivity of concrete mixtures. Another system developed by [Pigeon, M.; Toma, G.; Delagrave, A.; Bissonnette, B.; Marchand, J. & Prince, J. C. et al [2000], based on Kovler's system, and measured the stress due to restrained autogenous shrinkage. This system also used a computer controlled loading system. Deformation was measured using a direct current displacement transducer for greater precision. [Springenschmid, R.; Breitenbucher, R.; Mangold, M. et al [1995] developed the Temperature Stress Testing Machine (TSTM) to measure the tensile stress in concrete due to the heat of hydration. Attached to one end of a uniaxial concrete specimen was an adjustable crosshead. A computer controlled step motor applied a load to control the deformation of the concrete specimen as it reached a threshold of 0.001 mm (0.00004 in).
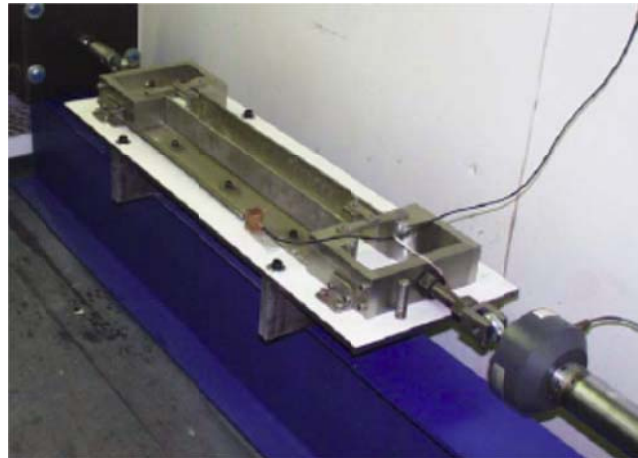
Figure 32. Active uniaxial restrained test [2002].

[Altoubat, S.A.; Lanage, D. et al [2002] developed a system currently in use at the University of Illinois at Urbana-Champaign that allows for simultaneous measurement of free shrinkage and deformation under restrained or constant tensile load. This test is also called the active uniaxial restrained test in which the length change due to shrinkage is recovered by applying external load to maintain the concrete sample at constant length (see Figure 33). The length change is measured by LVDTs, which is used as the control signal in this test. In such tests, the dog-bone geometry is used to grip the ends. To ensure a fully restrained test, the LVDT response to the loads and to shrinkage should reflect the deformation in the concrete sample. The study suggests mounting the LVDT to the concrete sample away from the grips to achieve a fully restrained test. This test was an improvement over the systems developed previously for several reasons. The 75x75 mm (3x3") cross-section was large enough to allow tests of concrete with 1" coarse aggregate. The applied load generated using a servo-hydraulic actuator had superior load stability and was capable of high load application. In addition, the revised deformation measurement technique avoided grip-specimen interaction, which caused inaccurate strain measurements in preliminary tests.
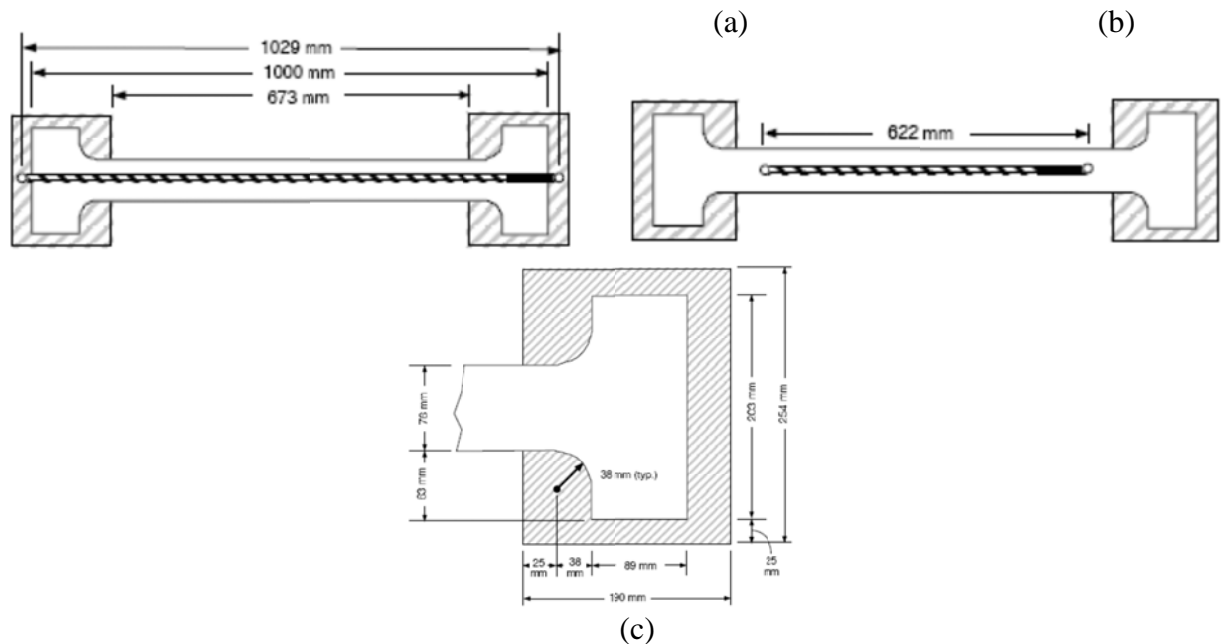
Figure 33. Two LVDT mounting methods. (a) LVDT mounted on grips and (b) LVDT
mounted on concrete. Grip detail given in (c) [2002].

Ring Tests

Several researchers have attempted to evaluate the shrinkage and cracking behavior
of concrete and cement based materials under restrained conditions. The most common
standard procedure is the restrained ring test. The ring test was designed to restrict
concrete shrinkage and induce cracking so that the cracking tendencies of different mixes
could be compared under similar conditions. [Weiss, W.J.; Shah, S.P. et al [2002] assumed
that the concrete ring simulated an infinitely long pavement restrained from shrinking freely.
[Krauss, P.D.; Rogalla, E. A. et al. [1996] summarized the usefulness of the ring test. First, it
is simple and the test apparatus is relatively inexpensive to construct. Analysis does not
require the use of complex calculations or assumptions of early-age concrete behavior. The
effects of stress development, volume deformation, and creep at early ages can all be
considered simultaneously, and the stresses resemble in-service stresses. Thus, the most
important factor is that all of the material variables affecting cracking can be evaluated
together using a single procedure. The ring test also produces easily visible cracks.

In the restrained ring test, a concrete ring is cast around an inner steel ring. The steel
ring restrains the shrinking concrete, producing an internal pressure on the concrete ring,
which causes tensile hoop stresses to develop in the concrete. When the tensile stresses
minus the relaxation due to creep exceed the tensile strength of the concrete, cracking will

100

occur. The steel ring can also be instrumented to monitor the strain development and determine time to cracking. The ring test is frequently preferred for quality control testing and material evaluation since it is relatively inexpensive to construct and does not have the problem associated with providing sufficient end restrain like linearly restrained shrinkage tests [Moon, J.; Rajabibour, F; Pearse, B.; Weiss, J. et at [1988].

Between 1939 and 1942 [Carlson, R.W.; Reading, T. J. et al [1988] performed the first restrained ring tests used to examine the influence of cracking resistance on shrinkage cracking in concrete walls. The authors cast a 25 mm (1 in.) thick concrete ring around a 25 mm (1 in.) thick, 175 mm (7 in.) diameter steel ring, as shown in Figure 34. The height of the specimen was 38 mm (1½ in.). Drying was limited to the outer circumference of the concrete ring by sealing the top and bottom surfaces. The rings were dried at relative humidities of 25, 50, or 75 percent. Time of cracking was determined by periodical visual observation. Companion free shrinkage bars were used to establish the strain at the time of cracking. These rectangular specimens had the same cross section as the ring and were 305 mm (12 in.) long. To simulate the shrinkage at the circumferential surface of the concrete ring, the free shrinkage specimens were sealed to allow drying from one side only. Free shrinkage measurements were made on the exposed surface, as well as the opposite surface. [Carlson, R.W.; Reading, T.J. et al (1988] used the strain from free shrinkage bars to determine the strain in the rings at time of cracking. They found that the stresses at the time of cracking for specimens dried in the harshest environment were the highest. In the harshest environment, the specimens also experienced shorter times to cracking.
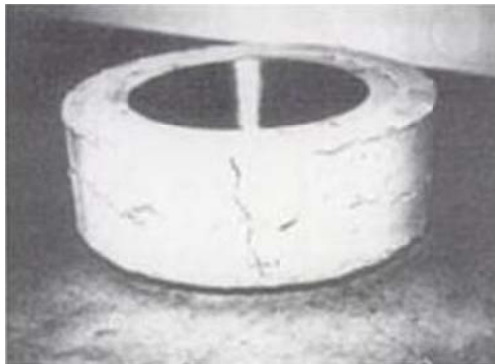

Figure 34. Ring test specimen by Carlson and Reading [1988].

Fiber reinforced concrete was tested by [Grzybowski, M.; Shah, S.P. et al.[1990] using a ring-type specimen to simulate restrained shrinkage cracking as shown in Figure 35. The authors studied two types of fibers (steel and polypropylene) in different amounts, at 40% relative humidity and 20ºC (68 F). The length of drying was 6 weeks and drying was only allowed from the outer circumferential surface. The width of the specimen, 140 mm (5.5 in) was four times the thickness (35 mm – 1.38 in). With this dimensions, the authors assumed uniform shrinkage along the width of the specimen. The top surface of the concrete ring was sealed with silicone-rubber and the bottom surface remained on the form to permit drying from the circumferential surface only and a cardboard tube was used as an outer mold. The inner circumferential surface of the concrete ring was sealed off using silicone-rubber as in the top surface. The rings were monitored with three strain gages that were attached to the outside of the concrete ring at mid-height. The authors found that fibers did reduce the crack widths, and steel fibers performed better than polypropylene fibers.
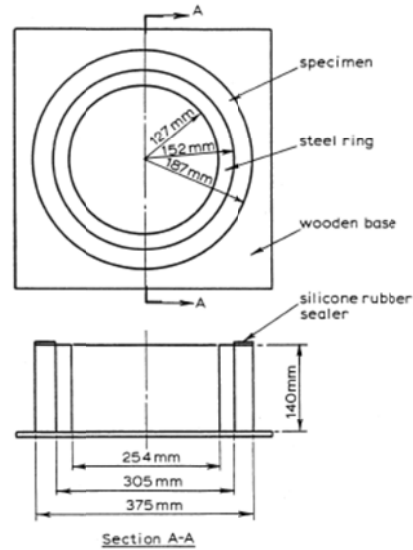
Figure 35. Restrained Shrinkage Test Specimen by Grzybowski et al. [1990].

In order to improve the fundamental understanding of information provided by the ring test [Hossain, A.B.; Pease, B.;Weiss, J. et al. [2006], investigated restrained shrinkage cracking in concretes with low water-cement ratios. The authors discussed the issues related to ring and free shrinkage specimen geometries, also described how the computed elastic and the actual residual stresses can be utilized together to obtain a measure of the stress relaxation that can be expected with each mixture. To better illustrate the micro-cracking and visible crack development process, authors performed acoustic emission testing. The experiments indicated that specimens with a higher level of restraint exhibited more micro-cracking as a part of stress relaxation process. The concrete in the rings authors tested had an inner diameter of 300 mm (12 in.), a height of 75 mm (3 in.) and an outer diameter of 450 mm (18 in.) as shown in Figure 36. To examine the degree of restraint provided by the steel rings, the authors used rings with thicknesses including 3.1 mm (1/8 in.), 9.5 mm (3/8 in.), and 19 mm (3/4 in.). Four strain gages were connected to each ring at mid height and interfaced with a data acquisition system. The concrete specimens were sealed for 24 hours and then demolded. After demolding, the outer circumference of the specimen was sealed using two layers of aluminum tape to allow drying from top and bottom surface of the rings and then placed in a constant RH (50%) and temperature 23 °C (73 °F). For comparison, rings were also tested with the top and bottom surfaces sealed to allow drying from the outer circumference only. Free shrinkage was also tested by using two different geometries: two mortar prisms of 75 mm (3 in.) square cross-section and 250 mm (10 in.) gage length according to ASTM C 157, dried from either two sides only or all sides and unrestrained ring specimens. The different drying regimes were studied to determine which one most closely matched the shrinkage of the unrestrained ring specimens. The authors observed that shrinkage increased as the surface to volume ratio of the specimens increased. The authors also proposed the calculation of *cracking potential*, expressed as the ratio of the residual stress in the concrete ring (calculated from the strain in the concrete and the geometry of the concrete and steel rings) to the time-dependent splitting tensile strength. Their results showed that failures occurred at cracking potentials between 0.7 for highly restrained specimens and 1.0 for lightly restrained specimens.
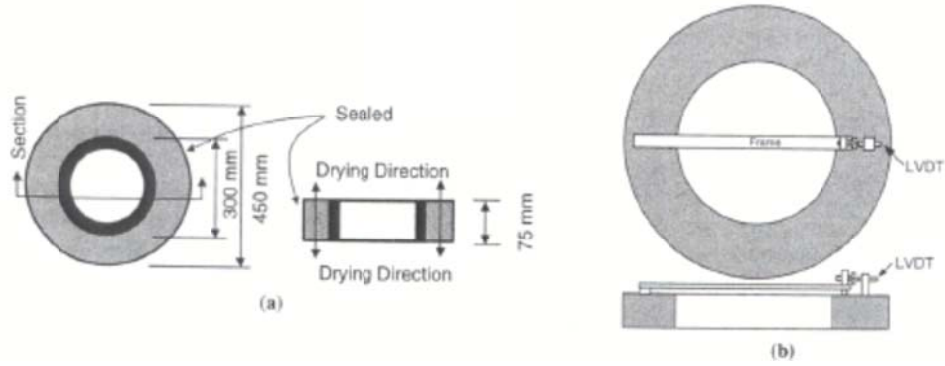
Figure 36. Ring specimens used by Hossain et al. (a) restrained ring geometry and (b) unrestrained ring specimen [1834].

Geometry of the Ring Test

The potential for cracking in portland cement based materials is strongly influenced by factors such as the magnitude of shrinkage, rate of shrinkage, stress relaxation, degree of structural restraint, age-dependent material property development, and structural geometry [Weiss, J.; Yang, W; et al., 1997]. Based on experimental results using a fracture-based failure criterion, [Weiss, W. J.; Yang, W.; Shah, S.P. et al., 2004], explained that specimens of different size and geometry exhibited a dramatically different restrained shrinkage cracking response. The authors used nine ring specimens for each concrete mixture with three different wall thicknesses (30, 75, and 150 mm). The inner radius was maintained constant at 150 mm as illustrated in Figure 37. According to the authors, experimental results and theoretical results using nonlinear elastic fracture mechanics-based theory showed that the age at which shrinkage cracking occurs is size/geometry dependent. [Weiss, et al., 1997] also found that as the material becomes less brittle, its geometry dependence increases.
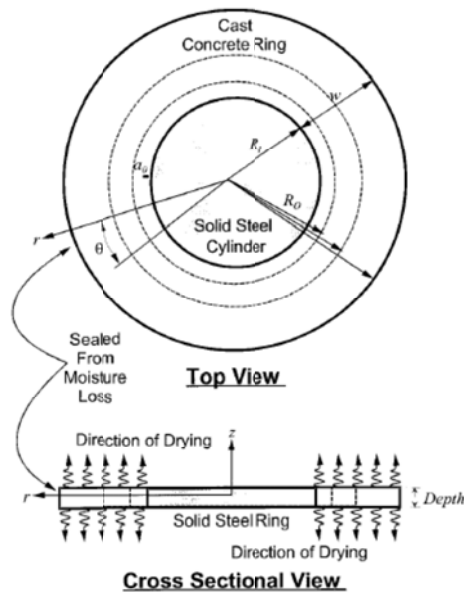

Figure 37. Specimen geometry and notation used by Weiss et al. [2004].

Other researchers such as [Krauss, P.D.; Rogallia, E.A. et al [1996] evaluated the geometry of both the steel and concrete rings by subjecting the concrete to either a uniform shrinkage stress or a shrinkage stress that increased linearly from the steel-concrete interface; simulating drying from either the top and bottom surface or the circumferential surface. They found that the concrete shrinkage stresses and cracking tendency were not significantly different for steel ring thicknesses between 13 mm (½ in.) and 25 mm (1 in.). They also observed increased steel stresses for thinner steel rings and increasing concrete stresses with larger ring diameters. As the height of the rings increased from 76 mm (3 in.) to 152 mm (6 in.), the concrete shrinkage stresses decreased. [See, H.T.; Attigobe, E.K.; Mittenbergre, M.M., et al [2003] used concrete rings where the height was four times the radial thickness and drying occurred on the circumferential surface, as shown in Figure 38. They assumed that drying shrinkage was uniform along the height of the ring because of this geometry. They also assumed that the concrete rings were under uni-axial tensile stress due to the internal pressure applied to the concrete by the steel ring. For a 152 mm (6 in.) tall, 12.5 mm (½ in.) thick steel ring and concrete rings with inner and outer radii of 165 mm (6½ in.) and 203 mm (8 in.), respectively, and subjected to an internal pressure, the hoop stresses at both the inner and outer edges were within 10% of the average hoop stress. The average radial compressive stress was about 10% of the average hoop stress, compared to about 25% for the dimensions of the AASHTO test. [See, H.T.; Attigobe, E.K.; Mittenbergre, M.M. et al [2003] also calculated the degree of restraint R, the ratio of the stiffness of the steel ring to the combined stiffness of the steel and concrete rings,

$$\overline{}$$

[1]

Where A_{st} and A_c are the cross-sectional areas of the steel and concrete, respectively, and E_{st} and E_c are the corresponding moduli of elasticity. Depending on the modulus of elasticity of the concrete, the authors calculated a degree of restraint of 70 to 75% for their setup. For the AASHTO geometry, the degree of restraint is between 55 and 60%, indicating that, under similar conditions, it will take longer for the rings to crack in the AASHTO test.
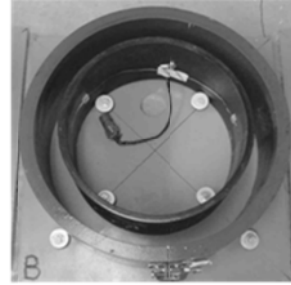

PLAN VIEW
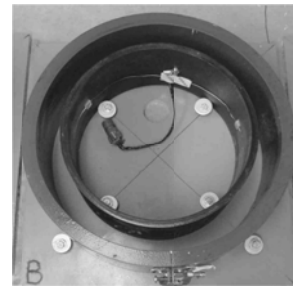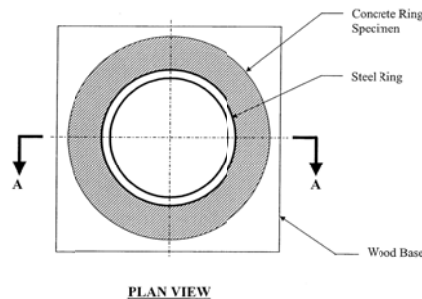
Figure 38. Dimensions of ring setup used by See et al. [2003.]

Kovler et al. [1993] determined that the time-to-cracking was related to the thickness of the concrete ring. Based on a ring test dried from the circumference of the concrete, they first established that the depth of drying increased proportionally with the square root of drying time. Through re-analysis of previous ring data, they also observed that the time-to-cracking was linearly proportional to the square of the concrete ring thickness. Combining these two observations, they concluded that the depth of drying at cracking was proportional to the thickness of the ring, and suggested that thicker rings could develop larger flaws before failure occurred.

To illustrate the influence of the ring dimensions on the potential for cracking, [Weiss, W. J.; Shah, S.P. et at [2002] compared two series of ring tests, a short and a tall ring series (see Figure 39). In both series, concrete rings were cast around a 150 mm (6 in.) diameter solid steel core. In the tall ring series, the height of the specimen was held at 150 mm (6 in.) while thicknesses of 25, 75, and 150 mm (1, 3, and 6 in.) were used to simulate varying slab thicknesses. These rings were exposed to drying from the outer circumference, causing a moisture gradient between the inner and outer edges of the concrete ring. These rings were dried from the top and bottom surfaces and experienced a uniform moisture gradient along the radial direction of the concrete ring. All specimens were cured at 30 °C (86 °F) for 24 hours and dried at 30 °C (86 °F) and 40% relative humidity. Companion 100 x 100 x 400 mm (4 x 4 x 16 in.) free shrinkage prisms were dried from two surfaces only. The results of the tall ring series showed that time to cracking was delayed as ring thickness increased. The short ring series showed that cracking potential decreased with increased ring thickness and that the use of shrinkage reducing admixtures delayed or prevented cracking. Authors concluded that thicker concrete sections are more resistant to cracking, with uniform moisture distribution and with moisture profiles.
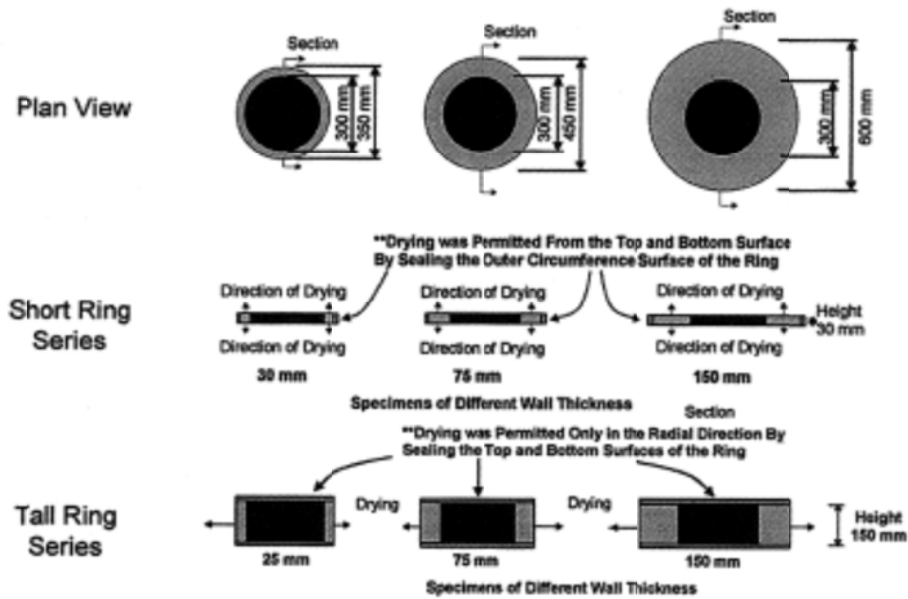
Figure 39. Specimen geometry for short ring and tall ring specimens by Weiss and Shah [2002].

According to [Kovler, K.;Sikuler, J.; Bentur, A. et al [1993] there are limitations in the restrained shrinkage tests by using conventional ring specimens with a steel core. It is because of their relatively low crack sensitivity, and as a result a test may need to be continued for several weeks before the presence or absence of cracks can be established with certainty. They attributed this low crack sensitivity to the geometry and test conditions which do not allow sufficient stress levels to develop, and to the viscoelastic response of the material which will cause some stress relaxation. In order to improve the crack sensitivity and achieve quicker results with the restrained ring test by replacing the inner steel ring with a material with a high coefficient of thermal expansion. They tested fiber-reinforced concrete rings with an outer diameter of 236 mm (9.3 in.) and a height of 43 mm (1.7 in.) as shown in Figure 18. The inner diameter of the rings ranged from 125 to 187 mm (4.9 to 7.4 in.). Increased crack sensitivity was achieved by casting the rings around a solid Perspex core, a Plexiglas material that has a higher coefficient of thermal expansion and lower modulus of elasticity compared to steel. The specimens were subjected to a temperature increase, which caused the Perspex core to expand and increase the stress in the concrete. By subjecting the specimens with a Perspex core to an increase in temperature, the authors were able to produce cracks in the hardened concrete in as little as 20 to 60 minutes. However, they were unable to produce cracks in rings subjected to drying immediately after casting. The authors then added a Perspex wedge to the outside of the Perspex core to produce a stress concentration. Cracks then appeared in the hardened concrete within 1 to 2 minutes of drying and in fresh concrete within 20 to 30 minutes. The authors concluded that cracking can be induced within a relatively short time, thus providing an effective test method which can yield useful data within a period of a few days or less.
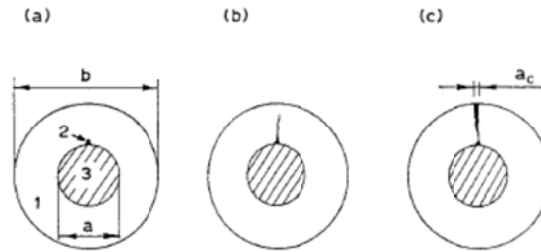
Figure 40. Schematic diagrams for (a) modified ring test (b) stabilized crack and (c) micro cracking across the ring: (1) concrete ring (2) stress raiser and (3) Perspex core [no date].

Standardization of Ring Tests

AASHTO Ring Test

Because of its simplicity and versatility, The American Association of State Highway and Transportation Officials AASHTO, recommended a version of the ring test as a standard test method that can be used for assessing cracking potential of various concrete mixtures that is now standardized under the designation T 334-08, Estimating the Cracking Tendency of Concrete [ASTM C 1581/C1581M-09 et al [2008]. This method covers the determination of the cracking tendency of restrained concrete specimens. The procedure determines the effects of variations in the properties of concrete as related to the time-to-cracking of concrete when restrained. ASHTO recommends a fixed geometry that allows drying from the outer circumference. This results in a complex stress field that varies its shape over time due to the presence of moisture gradients [Weiss, W. J.; shah, S.P. et al [2002]. Factors such as aggregate source and gradation, aggregate-paste bond, cement type, cement content, water content, mineral admixtures, fiber reinforcement, and chemical admixtures can be evaluated. The test consists of a concrete ring with inner diameter of 300 mm, and thickness and height of 75 and 150 mm, respectively. The restraining steel is a ring with a wall thickness of 12 mm. Strain gauge is attached to the inner surface of the steel ring and enables to calculate stresses in the steel as well as the age of cracking The outer forms are removed from the concrete 24 ± 1 hour after casting, and after curing, the top and bottom surfaces of the ring are sealed. The specimens are dried at 69.8 ± 0.9 °C (21 ± 1.7 °F) and 50 ± 4% relative humidity. Strain gage data is recorded every 30 minutes beginning as soon after casting as possible. Every 2 to 3 days, the rings are visually inspected for cracks. After the concrete cracks, the time-to-cracking is recorded and the crack width is measured at three locations along its length.
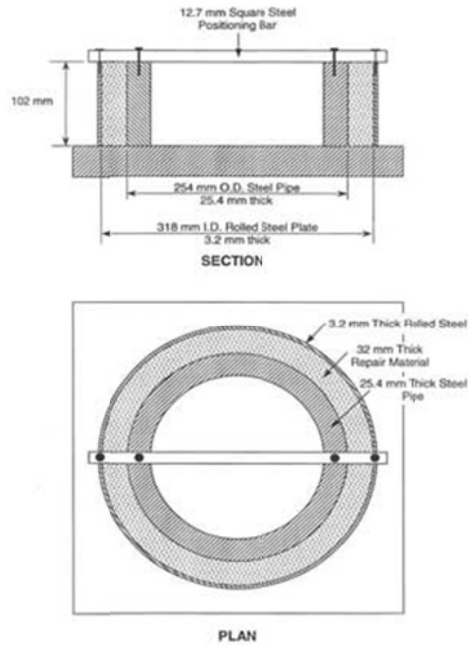
Figure 41. Test specimen mold recommended in AASHTO T 334-08
ASTM Ring Test.

The restrained ring test ASTM C1581-09 standardized by The American Society of Testing and Materials (ASTM) [Radlinska, A.; Bucher, B.; Weiss, J., et al [2008] covers the laboratory determination of the age at cracking and induced tensile stress characteristics of mortar or concrete specimens under restrained shrinkage. The procedure can be used to determine the effects of variations in the proportions and material properties of mortar or concrete on cracking due to both drying shrinkage and deformations caused by autogenous shrinkage and heat of hydration. According to [Radlinska, A.; Bucher, B.; Weiss, J., et al [2008], this is a simple, economical method to evaluate the susceptibility of a concrete mixture to exhibit shrinkage cracking. The geometry of the ring specimen is presented in Figure 42. The dimensions of the ring restrict the maximum aggregate size to 19 mm (3/4 in.). During the test, a sample of freshly mixed mortar or concrete is compacted in a circular mold around an instrumented steel ring. This test method is useful for determining the relative likelihood of early-age cracking of different cementitious mixtures and for aiding in the selection of cement based materials that are less likely to crack under restrained shrinkage. This test can also be used to determine the relative effects of material variation on induced tensile stresses and cracking potential. For materials that have not cracked during the test, the rate of tensile stress development at the time the test is terminated provides a basis for comparison of materials.
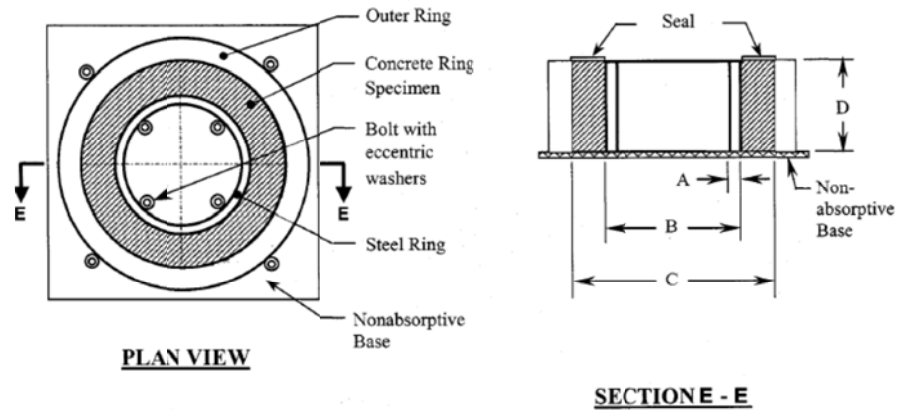
Figure Dimensions	Inch-Pound Units	SI Units
A	0.50 ± 0.12 in.	12.5 ± 0.13 mm
B	13.0 ± 0.12 in.	330 ± 3 mm
C	16.0 ± 0.12 in.	406 ± 3 mm
D	6.0 ± 0.25 in.	150 ± 6 mm

Figure 42. Test specimen mold recommended in ASTM C 1581 [no date].

SUMMARY

Ring-type tests are best suited to study the influence of concrete mixture temperature on the cracking tendency of concrete; various candidate concrete mixtures can be compared across the temperature regime of interest. However the conventional ring test would need to be modified to address issues related to placement of concrete at higher temperatures. The difference in coefficients of thermal expansion (CTE) of concrete and steel could induce additional stresses in the concrete ring that are not related to the field conditions. However, a lower CTE material that still has sufficient stiffness could be substituted in place of steel. It appears that the literature currently does not report studies that specifically used the ring test to evaluate the thermal cracking potential of concrete mixtures. The ring test could also be used for investigating the combined effects of concrete thermal and moisture shrinkage.

REFERENCES

AASHTO T-334-08, Standard Method of Test for Estimating the Cracking Tendency of Concrete.

ACI Committee 305, Hot Weather Concreting, American Concreting Institute, Farmington Hills, Michigan, 1999.

Altoubat S. A., Early Age Stresses and Creep-Shrinkage Interaction of Restrained Concrete, Ph.D. Thesis, Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, 2000.

Altoubat S. A., and Lange D., Grip-Specimen Interaction in Uniaxial Restrained Test, ACI SP-206, Concrete: Material Science to Application – A Tribute to Surendra P. Shah, 2002.

ASTM C1581/C1581M - 09, Standard Test Method for Determining Age at Cracking and Induced Tensile Stress Characteristics of Mortar and Concrete under Restrained Shrinkage

Attiogbe E. K., W. J. Weiss, and H.T. See, A Look at the Stress Rate Versus Time of Cracking Relationship Observed in The Restrained Ring Test, Advances in Concrete Through Science and Engineering, Northwestern University, Evanston, IL, Mar., 14 pp. 2004.

Bloom R. and A. Bentur, Free and Restrained Shrinkage of Normal and High-strength Concretes, ACI Materials Journal, Vol. 92, No. 2, pp. 211-217, 1995.

Brooks J., and A. Neville, Creep and Shrinkage of Concrete as Affected by Admixtures and Cement Replacement Materials, Creep and Shrinkage of Concrete: Effect of Materials and Environment, ACI SP-135, pp. 19-36, 1992.

Buil M., and J. J. Serrano, Effect of Fiber Addition on the Autogenous Shrinkage of Silica Fume Concrete, *ACI Materials Journal*, Vol. 86, No. 2, pp. 139-144, 1989.

Carlson R. W. and T. J. Reading, Model Study of Shrinkage Cracking in Concrete Building Walls, *ACI Structural Journal*, Vol. 85, No. 4, pp. 395-404, 1988.

Chariton, T. and W. J. Weiss, Using Acoustic Emission to Monitor Damage Development in Mortars Restrained from Volumetric Changes, Concrete: Material Science to Application, A Tribute to Surendra P. Shah, ACI SP-206, 2002.

Grzybowski M. and S. P. Shah, Shrinkage Cracking of Fiber Reinforced Concrete*, ACI Materials Journal*, Vol. 87, No. 2, pp. 138-148, 990.

Hossain A. B., B. Pease, and J. Weiss, Quantifying Early-Age Stress Development and Cracking in Low Water to cement Concrete, Restrained-Ring Test with Acoustic Emissions, Transportation Research Record 1834, 2005.

http://www.argenco.ulg.ac.be/etudiants/Multiphysics/Staquet_GrasMech_22042008-part11.pdf

Jensen O., and P. Hansan. A Dilatometer for Measuring Autogenous Deformation in Hardening Portland Cement Paste, Materials and Structures, 28, pp. 406-409, 1995.

Kovler K., J. Sikuler, and A. Bentur, Restrained Shrinkage Tests of Fiber reinforced Concrete Ring Specimens: Effect of Core Thermal Expansion, *Materials and Structures*, Vol. 26, No. 158, May, pp. 231-237, 1993.

Kovler K., Testing System for Determining the Mechanical Behavior of Early Age Concrete under Restrained and Free Uniaxial Shrinkage, Materials and Structures, V. 27, pp. 324-330, 1994.

Kovler K., and A. Bentur, Shrinkage of Early Age Steel Fiber Reinforced Concrete, *Archives of Civil Engineering*, Vol. 43, No. 4, 1997.

Kraai P., Proposed Test to Determine the Cracking Potential Due to Drying Shrinkage of Concrete, Concrete Construction, Vol. 30, No. 9, pp. 775-778, 1985.

Krauss P.D. and E.A. Rogalla, Transverse Cracking in Newly Constructed Bridge Decks, NCHRP Report 380, Transportation Research Board, National Research Council, Washington, D.C, 1996.

Mehta, P.K., Volume Change, STP 169C, Significance of Tests and Properties of Concrete and Concrete-Making Materials. Klieger and Lamond (eds.), pp. 219-228, ASTM, 1994.

Mehta P. K., Performance of High Volume Fly Ash Concrete in Hot Weather, Innovations in Design with Emphasis on Seismic, Wind, and Environmental Loading; Quality Control and Innovations in Materials / Hot Weather Concreting, Proceedings, ACI Fifth International Conference, Cancun Mexico, 2002.

Mindess J., F. Young, and D. Darwin, Concrete (2nd edition)*, Prentice Hall*, NJ, 2003.

Moon J., F. Rajabipour, B. Pease, and J. Weiss, Quantifying the Influence of Specimen Geometry on the Results of the Restrained Ring Test, Journal of ASTM International, Vol. 3, No. 8, pp. 1-14, 2006.

Neville A. M., Properties of Concrete, 4th. Edition, Longman, UK. 1996.

Ödman, S., Effects of Variations in Volume, Surface Area Exposed to Drying, and Composition of Concrete on Shrinkage, RILEM/CEMBUREAU International Colloquium on the Shrinkage of Hydraulic Concretes, Vol. 1, 20 pp., 1968.

Padron I. and R. F. Zollo, Effect of Synthetic Fibers on Volume Stability and Cracking of Portland Cement Concrete and Mortar, *ACI Materials Journal*, Vol. 87, No. 4, pp. 327-332, 1990.

Pickett G, Effect of Aggregate on Shrinkage of Concrete and Hypothesis Concerning Shrinkage, *Journal of the American Concrete Institute*, Vol. 27, No. 5, pp. 581-590, 1956.

Pigeon M., G. Toma, A. Delagrave, B. Bissonnette, J. Marchand, and J.C. Prince, Equipment for the Analysis of the Behavior of Concrete Under Restrained Shrinkage at Early Ages, *Magazine of Concrete Research*, Vol. 52, No. 4, pp. 297-302, Aug 2000.

Radlinska A., B. Bucher, and J. Weiss, Comments on the Interpretation of Results from the Restrained Ring Test*, Journal of ASTM International,* Vol. 5, No. 10, pp. 1-12, 2008.

Reichard T. W., Creep and Drying Shrinkage of Lightweight and Normal Weight tes, Monograph – United States Bureau of Standards, No. 74, 30 pp., 1964.

See H. T., E,K. Attiogbe, and M.A. Miltenberger, Shrinkage Cracking Characteristics of Concrete Using Ring Specimens, *ACI Materials Journal*, Vol. 100, No. 3, pp. 239-245, 2003.

Shaeles C. A. and K.C. Hover, Influence of Mix Proportions and Construction Operations on Plastic Shrinkage Cracking in Thin Slabs, *ACI Materials Journal*, Vol. 85, No. 6, pp. 495-504, Nov-Dec 1988.

Shah S. P., M.E. Karaguler, and M. Sarigaphuti, Effects of Shrinkage Reducing Admixtures on Restrained Shrinkage Cracking of Concrete, *ACI Materials Journal*, Vol. 89, No. 3, pp. 291-295, 1992.

Shah S. P., W. J. Weiss, and W. Yang, Shrinkage Cracking – Can it be Prevented?, *Concrete International*, Vol. 20, No. 4, pp. 51-55, Apr. 1998.

Springenschmid R., R. Breitenbücher, and M. Mangold, Development of Thermal Cracking Frame and the Temperature-Stress Testing Machine, Thermal Cracking in Concrete at Early Ages, Proceedings of the International RILEM Symposium, Ed. By R. Springenschmid, Munich 1994, pp. 137-144, 1995.

Tanabe T., et al., Creep, Shrinkage and Durability Mechanics of Concrete and Concrete Structures, Vol. 1, CRC Press, London, 2009.

Weiss J., and W. Yang, Restrained Shrinkage Cracking in Concrete, Proceeding of the Sixth International Purdue Conference on Concrete Pavement Design and Materials for High Performance, Purdue University, 1997.

Weiss W. J., W. Yang, and S.P. Shah, Influence of Specimen Size/Geometry on Shrinkage Cracking of Rings, *Journal Of Engineering Mechanics*, Vol. 126, No. 1, pp. 93-101, January 2000.

Weiss W. J. and S.P. Shah, Restrained Shrinkage Cracking : The Role of Shrinkage Reducing Admixtures and Specimen Geometry, *Materials and Structures*, Vol. 34, No. 246, pp. 85-91, March 2002.