A Crack Is Not a Crack: Thermal Cracking

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This article, which is the eighth article in this series, is focused on thermal cracking. To set the stage for our discussion, let us cover the basics first. Hydration of cement is an exothermic chemical reaction. Understanding how heat is generated and dissipated during the curing of concrete is essential to improve our ability to design and maintain durable concrete bridges.

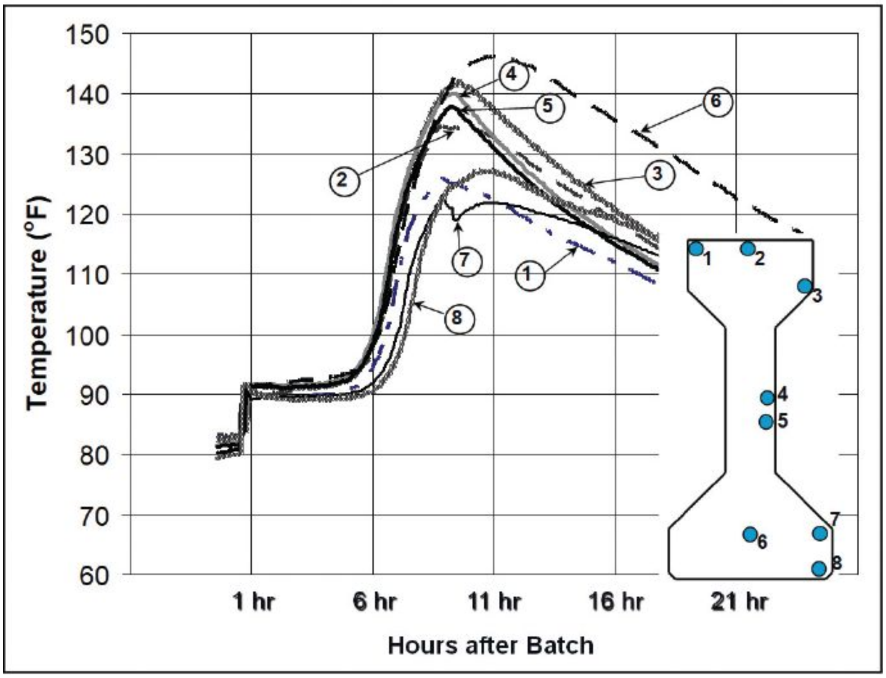
While this article is not solely directed toward exploring issues associated with mass concrete placement, let us start our discussion with a few values that relate to mass concrete. For most typical concrete mixtures, component dimensions larger than 36 to 48 in. would be classified as mass concrete. Project specifications and owners may stipulate requirements for the maximum temperature of the concrete core during curing, and the maximum temperature differential between the core of the concrete and its exposed surface in an effort to control and eliminate early-age cracking.

The maximum temperature of the concrete core used in mass concrete placement in Texas is typically 160°F.[1] The Texas Department of Transportation (TxDOT) specifies this limit to control thermal issues and delayed ettringite formation (DEF). DEF is an expansive mechanism that causes cracking similar to that caused by alkali silica reaction (ASR) (that is, map cracking). The difference between them is that ASR is associated with reactive aggregates, high-alkali pore solution, and presence of moisture in concrete while DEF is caused by aluminates in the cement. More specifically, the presence of excessive sulfates and alkalis within the cement, clinker, or aggregates may create a condition in which the sulfate is gradually released and bonds with aluminates. Excessive heat (temperatures in excess of ~160°F) creates a condition in which DEF potential is amplified. Conversely, controlling heat helps us mitigate/minimize the potential for DEF. With the description of ASR and DEF in place, let us now proceed to understanding the expansions associated with ASR and DEF. The maximum expansion potential and cracking severity associated with DEF are typically higher than those associated with ASR. The maximum temperature limit used in other states varies and can be as high as 180°F in some project specifications. Variations in the specified maximum temperature limits are rooted in the potential for DEF, and the presence or absence of ASR-reactive aggregates in the location under consideration. TxDOT specifies a maximum temperature differential between the core of the concrete placement and the exposed concrete surface of 35°F to minimize early-age thermal cracking. I will use the threshold values of 160°F and 35°F for maximum temperature and temperature differential, respectively, in the following discussion.

Precast Concrete Products

During his master's research at the University of Texas (UT), Tuchscherer[2] investigated the root causes of top flange cracking in AASHTO Type IV girders. While his research covered many topics and potential root causes, I will focus on one aspect of his work relevant to our discussion. In **Fig. 1**, the eight locations marked with blue dots within the cross section signify the locations where thermocouples were installed. As can be observed in the figure, the ambient

Figure 1. This plot shows the early-age temperature variation across a section that was part of an investigation at the University of Texas at Austin, into the root causes of top-flange cracking in AASHTO Type IV girders.[2]



conditions within the UT laboratory were such that the eight thermocouples initially measured temperatures ranging between 80°F and 84°F. The temperature of the fresh concrete mixture was around 90°F, and the temperature rose to a peak of 147°F around 11 hours after concrete was batched. This temperature is less than 160°F; therefore, DEF-related damage should not be an issue for this beam. The maximum temperature differential between the hottest and coolest location within the section is about 20°F to 25°F, which is less than the maximum temperature differential threshold value of 35°F used by TxDOT. The temperature-time history shown in Fig. 1 does not point to any temperature-related concerns. For all the test specimens in this study, we did not observe maximum temperatures or a temperature differential that would be a cause for concern.

As an aside, and perhaps an important one, we also tracked the mechanical properties of cylinders that were cured in a temperature regimen that matched the reference points shown in Fig. 1. Because cracking occurs when the tensile strength of concrete is reached, we were interested in seeing whether strengths differed between hotter and cooler locations. In short, the differences we saw in mechanical properties of concrete in hot and cool locations were comparable or less than the accepted strength variability values in the relevant ASTM specifications. While this observation is true for the AASHTO Type IV beams and for the concrete used in this study, it should not be extrapolated to all concrete components, geometries, and mixtures.

To improve our understanding of key factors at play, let us work on a few thought exercises. If the ambient conditions were different and if the constituent materials were warmer than the ones used in our study, the overall time history of the temperatures measured in eight locations would be different. For example, if the fresh concrete temperature were 10°F to 20°F warmer, we would expect to see the maximum temperature reached at each point to be higher, although the increases might not be exactly proportional for all points. This increase in fresh concrete temperature may have an impact on the temperature gradient. If warmer conditions were present and the maximum temperature limit of the project specifications were challenged, we would have to consider cooling down the fresh concrete mixture with ice or liquid nitrogen injection. If we could bring down the placement temperature of the fresh concrete mixture, that would go a long way toward controlling the temperatures experienced by the structural component.

The properties of the concrete mixture also play a key role in the temperature-time history. In our study, Type III cement was used in the concrete, as is typical in some precast concrete applications. The high early strength needed in precast, prestressed concrete applications is coupled with the elevated heats of hydration in the early stages. The use of supplementary cementitious materials (SCMs) such as fly ash and slag can offer an overall temperature-controlling benefit but will also slow strength gain. This delay in strength gain may present a significant challenge for precast, prestressed concrete applications. In cast-in-place concrete applications, particularly in substructure construction, the use of SCMs poses a lesser challenge because early strength gain is typically not as important as it would be in precast concrete production.

Bridge owners and the literature on mass concrete placement typically stay away from a strict definition for mass concrete. Technical reports[3–5] and project specifications typically define mass concrete as a large volume of concrete placement that will require management of the heat of hydration generated during curing. This broad definition is intended to recognize that component dimensions less than 36 to 48 in. may also need temperature management in cases where the proportion of cementitious material content is large. To aid in this technical challenge, TxDOT funded research at UT.[5,6] That research effort resulted in a publicly available software package (ConcreteWorks), which allows users to make multiple adjustments to manage heat generation and dissipation to minimize the risk of thermal cracking.

Case Studies

Large substructure elements such as drilled shafts, footings, and piers may fall into a category of mass concrete, which will require heat of hydration management. To better understand the underlying mechanics, let us consider a few case studies.

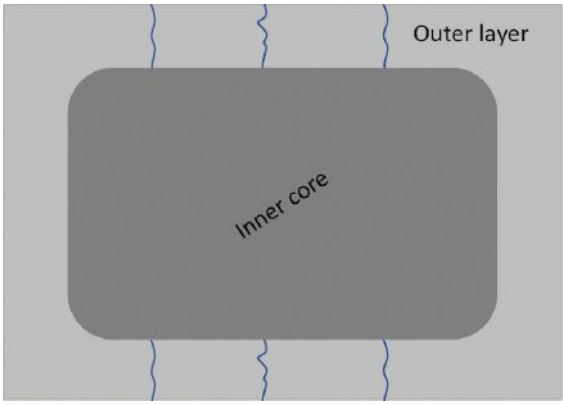
Large Pier

Let us consider a scenario in which the formwork for a hammerhead pier (**Fig. 2**) is removed early or a similar situation where the ambient temperature suddenly drops after the concrete for such a pier is placed. In this scenario, as the temperature of the concrete in the inner core continues to rise (or is maintained), the outer surface may suddenly cool down. The restraint provided by the inner core on the outer surface will result in the development of tensile stresses in the outer layer. When the tensile stresses in concrete reach the tensile capacity of the concrete, cracking in the outer layer will occur. The severity of this cracking will be a function of many factors, including the temperature gradient between the outer surface and inner core of the pier, the overall heat of hydration generated by the concrete mixture (which may or may not include SCMs), and environmental exposure conditions such as rapidly dropping or increasing ambient temperatures, just to name a few. In the event that such cracking occurs, the structural and durability implications of such cracks need to be determined. Remedial measures, crack repairs, installation of protective membranes, and other interventions may or may not be necessary.

Large Footing

Next, let us consider a large concrete footing supported on drilled shafts (**Fig. 3**). Suppose that a rain event rapidly cools the top side of the footing soon after the concrete is placed and the initial set occurs. With this cooling effect, let us assume that the temperature gradient becomes severe to the point where the

Figure 2. Schematic cross section of a column section in a hammerhead pier. Figure: University of Texas at Austin.



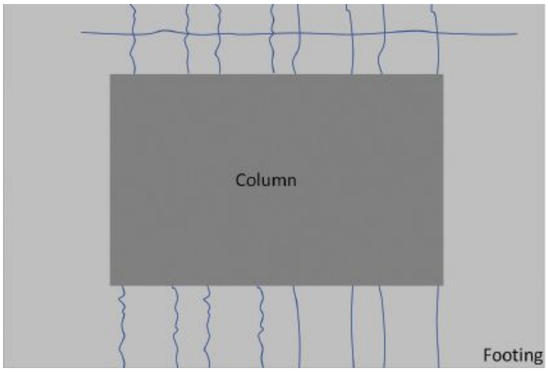


Figure 3. Schematic plan view of a single column–to–footing connection showing thermal cracking in the footing. Figure: University of Texas at Austin.

Figure 4. Cracking due to temperature gradients can be problematic with mass concrete placement. Figure: University of Texas at Austin.

difference between the hottest and coolest points in the footing becomes about 50°F. Due to this temperature differential, the cracks shown in Fig. 3 occur. Interestingly, this cracking resembles some of the cracking patterns we have discussed earlier in this series; similar map cracking may stem from ASR. However, cracking due to ASR will not be observed for days or weeks after the concrete is placed. If early-age map cracking is observed, it is probably not related to ASR. DEF cracks and restrained shrinkage cracks may also take the form of map cracking. Let us remember, a crack is not a crack. We must understand the root cause.

An examination of temperature-time histories of the footing can help us determine what may have happened. In addition, petrographic examination of a few cores taken from the footing could provide additional information to identify the root cause of the cracking. When cracking is due to thermal effects, an engineer must evaluate the structural and durability implications for the concrete component and invoke the use of remedial measures to restore the intended service life and structural capacity.

Figure 4 shows an example of thermal cracking in a real-world scenario. If conditions are not well managed, this type of cracking can be quite common; therefore, owners typically require management plans for mass concrete placement. One of the key takeaways from this discussion is that thermal cracking– and temperature gradient–related cracking issues cannot be directly tied to a single dimensional value such as 3, 4, or 5 ft. Depending on all the factors that surround heat generation during hydration and heat dissipation during curing, thermal cracking issues may be associated with dimensions that are different than the dimensions given in general guidance for mass concrete. ConcreteWorks is a commonly used software resource for those who are working on such thermal cracking issues and developing management plans. Management of heat of hydration, curing temperature, and dissipation is likely the most effective way of dealing with thermal cracking–related issues in concrete. Prevention, as always, is worth its weight in gold. Once the thermal cracking occurs, owners may incur additional expenses to rout and seal the cracks, apply coating on the structural component, invoke additional inspections, and so on. All these items can be costly and difficult to perform, and they may directly influence the service life of the concrete component and therefore the bridge.

Predicting the service life of a concrete component with thermal cracks is not a simple endeavor. Most service-life prediction models are based on diffusion of chemical solutions in porous media. Once cracks form in the concrete, the fundamental assumptions that are inherent to service-life prediction models come under question. Depending on their location, width, and depth, cracks in the concrete may provide direct conduits for chlorides to reach the reinforcing steel. Action plans for addressing thermal cracks in concrete will relate to the reinforcement type (for example, coated or uncoated steel, galvanized or pure stainless steel), as well as the environmental conditions. Once again, not all cracks are created equal. Understanding the root cause and the overall structural and durability context is key to making decisions that will be effective.

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