Hotter Cements, Cooler Concretes

Finer cements may require engineered solutions to reduce the maximum temperature rise in concretes

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Many things have changed in the concrete industry during the past 50 years, including the basic characteristics of portland cement—the main component of concrete's binder phase. In general, phase composition, fineness, and alkali levels have each changed in a manner that results in higher early strengths and higher early-age heat generation (so-called "hotter cements"). The objective of this article is to examine trends in cement production, and provide practical solutions to producing "cooler concretes" with "hotter cements."

Trends in cement production have mainly been driven by the desire for generating high early strengths to support fast-track construction. To a large extent, this has been achieved by finer grinding of the cement to enhance early-age hydration and strength development. Figure 1 presents two sets of survey data concerning cement fineness—the first showing mean fineness values from three surveys as presented by the Portland Cement Association[1] and the second based on individual results from the Cement and Concrete Reference Laboratory (CCRL) (**www.ccrl.us**) proficiency sample program compiled during the past 40 years.

While one can argue that there is no guarantee that the cements selected by CCRL are representative of the cements available from the industry as a whole, taken together, all of the results in Fig. 1 clearly indicate the trend of increasing cement fineness. Both regression lines in Fig. 1 have a statistically significant non-zero slope that indicates an increase in fineness of about 10 m^2/kg every 10 years. It may have been unlikely that this trend could continue, as the AASHTO M85-04 specification contained a maximum fineness limit of 420 m^2/kg for Types I, II, IV, and V cements. However, when the ASTM International and AASHTO standards were recently harmonized in 2009, the only maximum fineness limit that was established was a value of 430 m^2/kg for Type II(MH) and Type IV cements, with no maximum limits being established for Type I, II, or V cements.[2]

While increased fineness produces increased early-age strengths, it also produces a greater heat release at early ages—both due to increased rates of hydration. To demonstrate this nearly one-to-one relationship, Fig. 2 presents mortar cube compressive strength and isothermal calorimetry results, both for ordinary portland cement mortar mixtures prepared with six different fineness cements—all based on the same clinker—with finenesses ranging from 613 to 302 m^2/kg.[3] The mortar cubes for compressive strength were demolded after 1 day and stored in saturated limewater at 73.4°F (23°C) until testing. For the three ages examined in the study, as fineness increases proceeding from left to right in the graph, there is nearly a linear relationship between heat release and strength. At each of these three ages, finer cements are stronger, but they are also hotter.

While it seems clear that cements are getting hotter, the natural follow-up question is whether this is cause for concern. If a wide range of cement finenesses were available, the answer would be perhaps not. Because most cement producers are hesitant to produce a wide range of products, however, the same Type I/II cements that

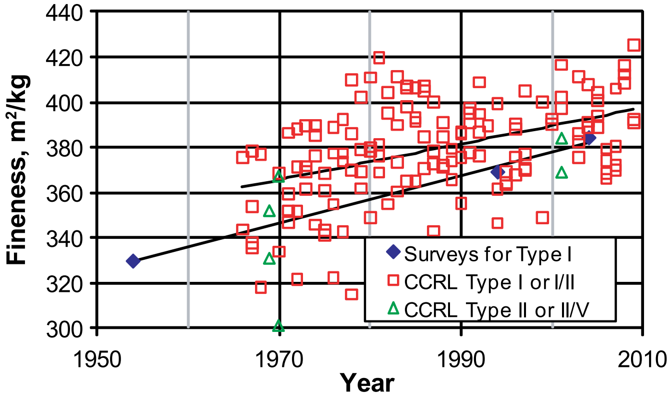

Fig. 1: Changes in Blaine fineness of cements from the 1950s to the present day. Individual regression lines are provided for the Type I survey and CCRL Type I or I/II data sets only

are manufactured for high early-age strength applications, such as high-rise construction, are also being employed in pavements and bridge decks, where long-term durability is often more critical than early-age strength. ASTM C150 specifically includes a low heat of hydration cement designated as Type IV (≤ 250 J/g cement for 7 days heat of hydration); to the authors' knowledge, there are currently no Type IV cements being produced in the U.S. In addition, for Type II cements, there is a low heat of hydration option that can be specified (≤ 290 J/g cement for 7-day heat of hydration); this approach is now employed in practice by state transportation agencies and other specifiers.

In fairness, it should be noted that some coarser cements are locally available, such as the 302 m²/kg cement (Fig. 2) currently produced in California. When available, such a coarser cement can be effectively blended with a finer cement to produce a range of cement finenesses and achieve a wide range of strength and heat release values as indicated in Fig. 2.[3] In many parts of the

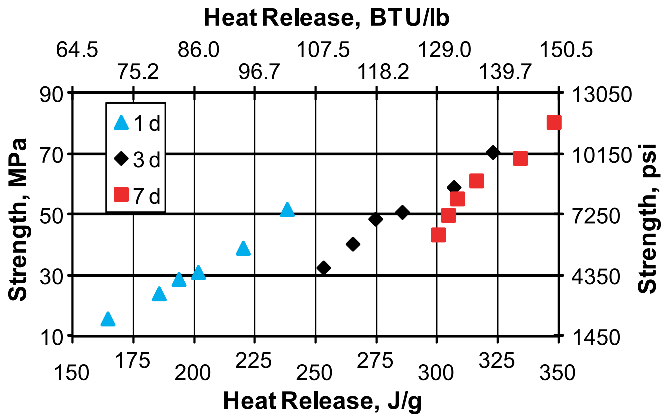

Fig. 2: Mortar cube compressive strength versus heat release at three different ages for mortars prepared with different fineness cements.[3] For each data series, fineness increases from left to right, with measured average particle sizes of 17.8 μm, 12.5μm, 11.2 μm, 9.1 μm, 7.9 μm, and 6.8 μm (1 μm = 3.9 × 10⁻⁵ in.)

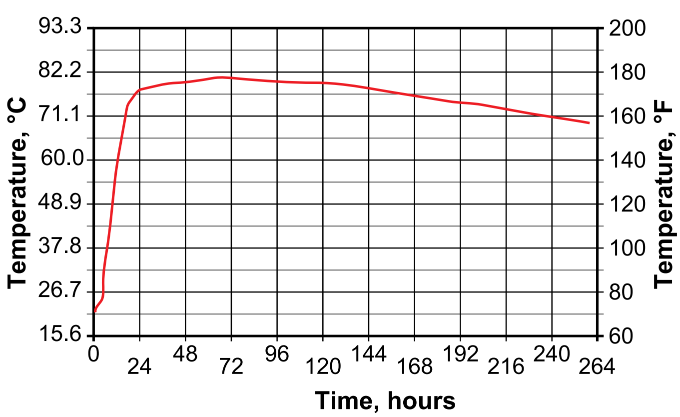

Fig. 3: Mass concrete core temperature

U.S., it's difficult to find a cement with a fineness of less than 360 m²/kg. Given that coarser cements are not readily available, the next question is whether the use of finer cements produces adverse effects in concrete. When high temperatures are produced in a curing concrete element, three issues of major concern are thermal cracking, reduced ultimate (in-place) strengths, and delayed ettringite formation (DEF).[4]

A REAL-WORLD EXAMPLE

In fall 2006, the Port Authority of New York & New Jersey (PANYNJ) was assigned the task of building the Freedom Tower—now called Tower One—at the World Trade Center site in New York City. The tower foundations, which required 12,000 and 8500 psi (83 and 59 MPa) concretes, were under construction at the time. The cementitious material contents for the 12,000 and 8500 psi (83 and 59 MPa) mixtures were respectively 1250 and 1040 lb/yd³ (742 and 617 kg/m³), and the portland cement contents were respectively 700 and 520 lb/yd³ (415 and 309 kg/m³). The foundations exhibited significant cracking within the first 48 hours. The internal temperature of an 8500 psi (59 MPa) foundation placement made in November 2006 was monitored and found to have reached about 170°F (77°C) at 24 hours, remaining above 160°F (71°C) for more than a week (Fig. 3). PANYNJ was alarmed that such a high temperature had been reached in November. What would the internal cure temperatures be in the summer months?

The tower core and structural walls for the tower are up to 6 ft (1.8 m) thick. For these, the concrete temperature rise was restricted to 160°F (71°C) to minimize strength reduction and the potential for DEF; the temperature difference between the core and surface temperatures was restricted to 35°F (19°C). The tower core required a 14,000 psi (97 MPa) mixture. Before the mixture was approved, 6 x 8 x 10 ft (1.8 x 2.4 x 3 m) blocks had to be placed and the temperature rise recorded, with companion test cylinders being made for determining strength and modulus of elasticity. The evaluated concrete mixture proportions and mechanical properties measured at 56 days are given in Table 1.

The core temperature profiles for the mixtures are given in Fig. 4. Mixture 1 had a cement content of 695 lb/yd³ (412 kg/m³) and reached a peak temperature of 182°F (83°C) in 24 hours. Mixture 2 had 264 lb/yd³ (157 kg/m³) of cement and reached a peak temperature of 162°F (72°C) in 40 hours. Based on these results, Mixture 2 was selected.

After about 6 months, 4 in. (102 mm) diameter cores were taken from the center of the blocks and 1 ft (0.3 m) from an outside face by vertically drilling from the top. Horizontal cores were also taken. All of the core results

TABLE 1:
MIXTURE PROPORTIONS FOR PANYNJ CONCRETES EVALUATED FOR THE CORE OF TOWER ONE

	Mixture 1	Mixture 2
Cementitious factor	1029 lb/yd³ (610 kg/m³)	723 lb/yd³ (429 kg/m³)
Cement factor	695 lb/yd³ (412 kg/m³)	264 lb/yd³ (157 kg/m³)
Slag	—	324 lb/yd³ (192 kg/m³)
Fly ash, Class C	301 lb/yd³ (179 kg/m³)	114 lb/yd³ (68 kg/m³)
Silica fume	33 lb/yd³ (19.6 kg/m³)	21 lb/yd³ (12.5 kg/m³)
Sand	1375 lb/yd³ (816 kg/m³)	1428 lb/yd³ (847 kg/m³)
Stone—57/67*	1080 lb/yd³ (641 kg/m³)	1162 lb/yd³ (689 kg/m³)
Stone—8	450 lb/yd³ (267 kg/m³)	497 lb/yd³ (295 kg/m³)
Water-cement ratio	0.25	0.31
Air content, %	1.9	3.0
Slump	10.75 in. (273 mm)	8.25 in. (210 mm)
Maximum core temperature	182°F (83.3°C)	162°F (72°C)
56-day compressive strength	18,140 psi (125 MPa)	17,290 psi (119 MPa)
56-day elastic modulus	7.81×10^6 psi (54 GPa)	8.31×10^6 psi (57 GPa)

*Aggregate size number per ASTM C33

for a given mixture were very consistent, as indicated by their respective strength ranges. For Mixture 1, the compressive strengths ranged from 12,320 to 12,880 psi (85 to 89 MPa) with an average of 12,640 psi (87 MPa). This was 30% lower than the 56-day results for corresponding cylinders cast on the day of the placement and cured in accordance with ASTM C31. The cores from Mixture 2 averaged 15,780 psi (109 MPa) with a range of 15,470 to 16,380 psi (107 to 113 MPa). This was 9% lower than the 56-day cylinders cast on the day of the placement.

The strength differences between cores and cast cylinders were attributable to three potential primary factors: consolidation, damage to cores during drilling, and the curing temperature during the initial 24 hours. Consolidation should not have been a significant factor because the concrete blocks contained no reinforcement and the mixture was placed at about an 11 in. (280 mm) slump. To assess how much reduction in strength was potentially due to coring for a nominally 14,000 psi (97 MPa) concrete mixture, PANYNJ cast 10 in. (254 mm) slabs in the laboratory with companion cylinders being cast; both cylinders and slabs were cured in the laboratory in accordance with ASTM C31. Four inch (102 mm) diameter cores were taken from the slabs at 7 and 56 days. The cores taken at 7 days were also cured in accordance with ASTM C31. The 56-day compressive strengths produced by the cores and companion cylinders were virtually the same. Therefore, the lower core strengths taken

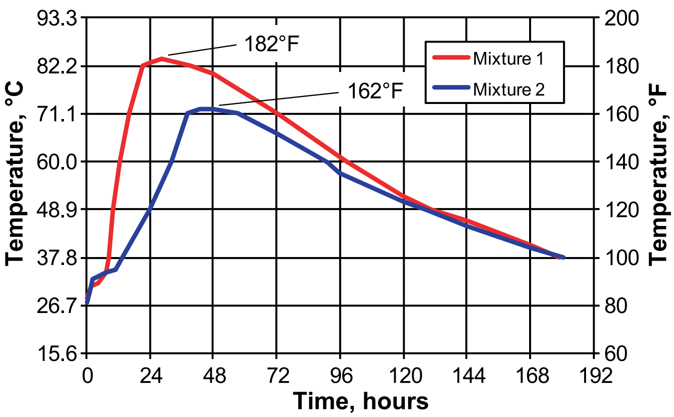

Fig. 4: Core temperatures of 14,000 psi (97 MPa) concrete mock-ups

from the original mock-up were not due to coring issues and were most likely due to the higher temperatures in the blocks (as the concrete was not cured in accordance with ASTM C31).

The adverse effects of high curing temperatures during the initial week should be considered when specifying high-strength and/or mass concrete. This is in agreement with the Portland Cement Association's *Design and Control of Concrete Mixtures*, which states, "If the initial 24-hour curing is at 100°F (38°C), the 28-day compressive strength of the test specimens may be 10 to 15% lower than if cured at the required ASTM C31 curing temperatures."[5]

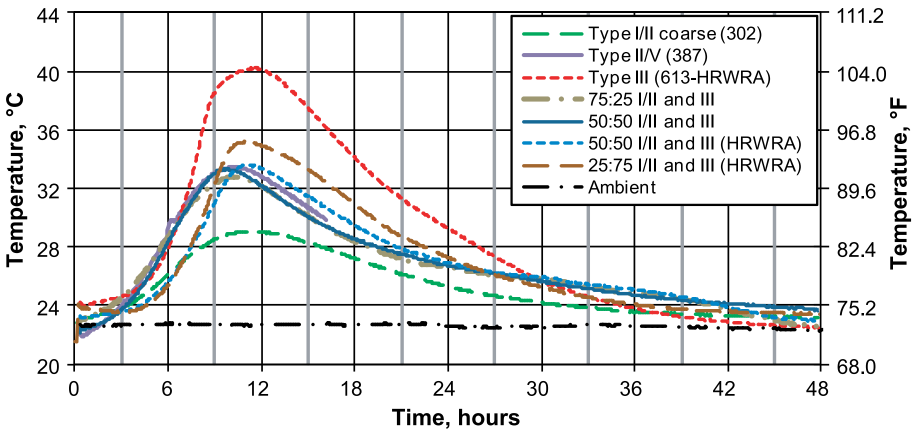

Fig. 5: Semi-adiabatic temperature versus time for mortars prepared with different fineness cements.[3] For the first three entries in the legend, the number in parentheses corresponds to Blaine fineness of cement in m²/kg. HRWRA indicates high-range water-reducing admixture

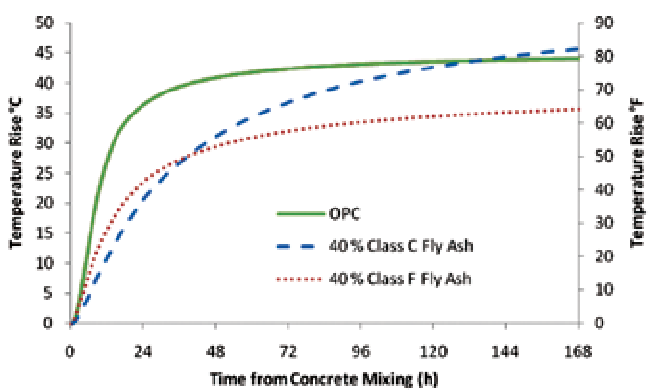

Fig. 6: Calculated adiabatic temperature rise of concretes made with and without fly ash

APPROACHES TO COOLER CONCRETES

Materials selection

Coarser cements: As mentioned previously, coarser cements can significantly reduce the rate of heat generation and the temperature rise within field concretes. For example, Fig. 5 provides semi-adiabatic calorimetry measurements of temperature rise for mortars prepared with different fineness cements, all containing 55% silica sand by volume and with a water-cement ratio (*w/c*) of 0.4.[3] The maximum temperatures are about 104, 91, and 84°F (40, 33, and 29°C) for the Type III, II/V, and coarse I/II cements, respectively. In addition to the inherent hydration rates of the cement, temperature rise in placed concrete will depend on specimen size, mixture proportions, aggregate thermal properties, and environmental boundary conditions. The latter three of these will be discussed subsequently in more detail. While cement fineness clearly has a large influence on heat generation and temperature rise of concrete, current market trends are such that switching to a coarser cement is not a viable option in many locations within the U.S.

Cement substitution: Partial cement replacement by supplementary cementitious materials (SCMs) has been used to limit the temperature rise in concrete construction for almost 100 years. Today, fly ash and slag are the most commonly used SCMs to control heat of hydration. The fly ash and slag will, like portland cement, release heat when they react, but they release a different total amount of heat and at a different rate than portland cement. Figure 6 shows the temperature rise as calculated from semi-adiabatic calorimetry[6] for concrete with a water-cementitious material ratio (*w/cm*) of 0.44 with 40% of the Type I/II ordinary portland cement (OPC) replaced with either a Class F fly ash or a Class C fly ash. Although for this mixture, the Class C fly ash ultimately releases more heat than the portland cement mixture, the rate of heat release is much lower, which can significantly reduce the risk of early-age cracking.[7]

High volumes of cement replacement by SCMs (greater than 50%) are also routinely used to control the temperature rise in concrete, with excellent results possible. It should be noted that concrete mixtures with high volumes of SCMs should be wet cured for longer periods of time than OPC mixtures, and should generally not be used in structural members exposed to deicing salts. Additionally, potentially detrimental SCM-cement-admixture interactions should be investigated up front in trial mixtures using calorimetry and other appropriate analytical methods.[8,9]

When considering specific SCMs, the use of fly ash has slowly gained acceptance in the last 20 years. More recently, due either to the green movement or to the construction of larger concrete elements that are considered to be mass concrete, the replacement of cement by fly ash has progressed from 20% to up to 50% by mass. It is now quite common to see a specification allowing a fly ash replacement ratio of up to 50% for foundations and other encased structures. Most flatwork applications allow the use of 20 to 35% replacement. In hot weather operations, these levels of replacement can reduce or eliminate the need for set-retarding admixtures that may have an undesirable effect on the finishability of the concrete. In selected geographical areas where it's economically feasible, the use of slag up to 65% by mass is often allowed.

In mass concrete applications, the most direct way to reduce the internal heat of hydration to prevent thermal cracking and DEF is through the use of high volumes of pozzolans to replace cement. The concrete producer has to balance the service-life requirements of the designing engineer, to ensure the durability of the structure, with the constructibility requirements of the contractor—to safely remove forms and shoring in a timely manner. In current practice, fly ash at a 40% by mass replacement ratio is often employed.

Early in 2010, for a highway project for the Texas Department of Transportation and the North Texas Tollway Authority that included many mass concrete structures, "cool concrete" was produced using Class C fly ash. An elevated bent cap was built with a span of 100 ft (30.5 m) and a cross section of 8 ft 8 in. (2.6 m) by 8 ft 4 in. (2.5 m), using 267 yd^3 (204 m^3) of concrete with a 40% replacement ratio of Class C fly ash. The maximum core temperature was well under the specification limit of 160°F (71°C) (Fig. 7), and the constructibility requirements from the contractor were successfully met.

The use of limestone fillers in place of up to 20% portland cement in concrete has been recently promoted to reduce the heat of hydration and the carbon footprint of concrete.[10] Because the reactivity of the limestone fillers is generally minimal, these substitutions are most effective in concretes with a low *w/cm*. Because space restrictions prohibit complete hydration for mixtures with *w/cm* < 0.4, a portion of the cement essentially functions as inert filler. This is a modern version of historically successful techniques for controlling the concrete heat of hydration. In fact, fine granite fillers were successfully used in a blended cement in the construction of the Elephant Butte Dam in 1915.[4] When cement replacements are employed, one must keep in mind that these potential reductions in heat generation per unit volume of concrete will be partially offset by any reductions in *w/cm* that are obtained via an increase in the cementitious materials content of the concrete mixture. It's the overall cement content of the concrete mixture that is most critical in determining its potential for heat generation, as opposed to the level of cement replacement that has been achieved.

Aggregate selection: Aggregate selection can have a major impact on the cracking tendency of the concrete. Because the aggregates make up a majority of the concrete volume, the concrete thermal properties are primarily determined by the thermal properties of the aggregates. The three most important concrete thermal properties are its specific heat, thermal conductivity, and coefficient of thermal expansion (CTE).[11] A low specific heat means that for the same cementitious materials used, the concrete will have a higher temperature rise from the heat of hydration. A low thermal conductivity will result in the concrete core taking longer to cool down. The selection of aggregates with a low CTE will result in lower tensile stresses in the concrete and ultimately less cracking. Table 2 provides typical values of thermal properties of commonly employed concrete aggregates; Table 3 provides typical values of thermal properties for concretes prepared with them.

Mixture proportioning

As pointed out by Mather, using less cement can be beneficial in applications where durability is equally as or more important than strength.[12,13] To reduce cement content, PANYNJ has been a proponent of blending aggregates in a concrete mixture to produce a more uniform combined aggregate gradation and of using a larger nominal maximum aggregate size. By increasing the nominal maximum stone size and using a well-graded combined aggregate, PANYNJ has been able to decrease the cement and cementitious contents, reducing potential thermal cracking. Reducing the paste content has also reduced the potential for shrinkage cracking, allowing a concurrent increase in the transverse joint spacing pattern.

In 2010, PANYNJ repaved a major runway at JFK International Airport with about 250,000 yd^3 (191,000 m^3) of concrete. The Federal Aviation Administration (FAA) P-501 specification requires a minimum cementitious material content of 564 lb/yd^3 (335 kg/m^3). PANYNJ obtained approval to use 550 lb/yd^3 (326 kg/m^3) maximum cementitious material with a 40% slag substitution for cement. This reduction in cementitious content is made possible by the requirement to use a 2 in. (50 mm) nominal maximum size stone, a minimum 70% total aggregate content, and a combined aggregate gradation supported by the 0.45 power chart for coarseness and workability factors.

The FAA P-501 specification requires a flexural strength of 650 psi (4.5 MPa). To date, the concretes have

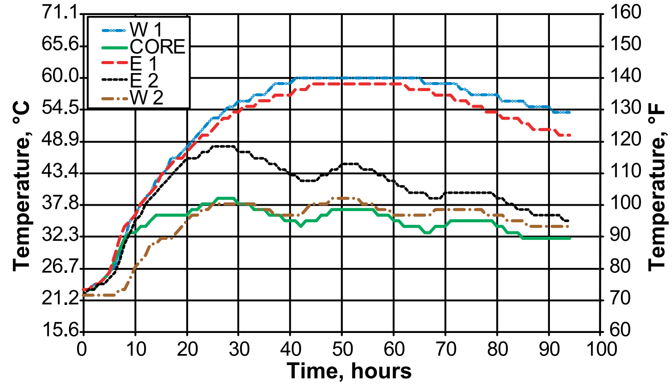

Fig. 7: Measured concrete temperatures for Texas highway mass concrete project. In the legend, E and W refer to the East and West sides of the column, with E2 and W2 being above E1 and W1 (in the coil projections of the column), respectively

TABLE 2:
Common thermal properties for different aggregates

Aggregate	Thermal conductivity of aggregate, BTU/(h·ft·°F) (W/(m·K))	Specific heat of aggregate, BTU/(lb·°R) (J/(g·K))	Coefficient of thermal expansion of aggregate, millionths/°F (millionths/°C)
Siliceous river gravel	2.9 to 4.6 (5 to 8)[11,16,17]	0.18 (0.75)[18]	5.6 to 6.7 (10 to 12)[11]
Limestone	1.2 to 1.7 (2 to 3)[11,17,19]	0.2 (0.84)[18]	1.9 to 3.3 (3.5 to 6)[11]
Dolomitic limestone	2.3 to 2.9 (4 to 5)[11]	0.2 (0.84)[20]	3.9 to 5.6 (7 to 10)[11]

TABLE 3:
Common thermal properties for concrete made with different aggregates

Aggregate	Thermal conductivity of concrete, BTU/(h·ft·°F) (W/(m·K))	Specific heat of concrete, BTU/(lb·°R) (J/(g·K))	Coefficient of thermal expansion of concrete, millionths/°F (millionths/°C)
Siliceous river gravel	1.8 to 2.4 (3.1 to 4.1)[21]	0.23 (0.97)[22]	6 to 7 (10.8 to 12.5)[22-24]
Limestone	1.3 to 1.9 (2.2 to 3.2)[21]	0.25 (1.04)[22]	1.9 to 3.3 (3.5 to 6)[25]
Dolomitic limestone	1.9 (3.3)[21]	0.23 (0.97)[22]	2.9 to 5.5 (5.3 to 9.9)[22,23]

been averaging a 28-day flexural strength of over 1000 psi (6.9 MPa). Furthermore, PANYNJ has been able to increase joint spacing to 25 ft (7.6 m), beyond the FAA requirement of 20 ft (6.1 m), and the in-place concrete has experienced very little cracking.

Through the use of an intermediate size lightweight aggregate (LWA), ASTM C330 3/8 in. to No. 8, TXI has been able to improve the total gradation of a typical concrete mixture that contains ASTM C33 No. 57 crushed limestone stone and natural sand and would otherwise normally be considered as gap graded. The improvement in the total gradation of the concrete mixture with this hybrid aggregate system results in enhanced workability of the concrete, obtaining the desired slump with a reduced water content and ultimately producing higher strength concrete, or allowing a reduction of the cementitious materials content of the concrete mixture, depending on the specific requirements of the project. In addition, the use of an intermediate prewetted LWA provides the benefits of internal curing, further increasing the strength of the concrete.[14] When this allows for a reduction of the cementitious content of the mixture, a "cooler concrete" is produced. As reported by Villarreal and Crocker,[14] "Production mixtures that include lightweight aggregate typically produce compressive strengths that are about 1000 psi (6.9 MPa) higher than similar mixtures without lightweight aggregate."

Construction scheduling

Adjustments to the construction schedule can be made to reduce the risk of cracking in the concrete, especially for pavements and bridge decks. Before the concrete sets, the stresses that develop in the concrete are negligible. When concrete setting occurs at the same time that the concrete temperature peaks and then begins to decrease, the concrete will immediately generate tensile stresses, increasing the risk of cracking. If, however, the concrete sets as the concrete temperature is beginning to rise, and if the concrete is restrained from expanding, the concrete will go into compression, concurrently decreasing the risk of cracking.

To illustrate this point, the temperature development within a 10 in. (254 mm) thick concrete pavement was modeled using the ConcreteWorks software package (**www.texasconcreteworks.com**) for a placement on August 17, 2006, at 10:00 a.m. and at 10:00 p.m. in Austin, TX. The predicted concrete pavement surface temperatures are shown in Fig. 8. Concrete stresses and cracking risks were then calculated using the HIPERPAV III software package (**www.hiperpav.com**) and are shown in Fig. 9.

The analysis shows that concrete placed at 10:00 a.m. sets just as the sun sets and the nighttime temperature drop occurs, cooling the concrete and causing a significant temperature drop. The concrete mixture placed at 10:00 p.m. will set as the sun is rising, causing the concrete to heat up and develop further compressive strength, giving a reserve stress capacity for when the concrete eventually cools the following night. These freely available computer programs can help the contractor better understand the complex interactions between the concrete's heat of hydration; modulus and strength development; and its boundary conditions as determined by the environment, curing conditions, and member geometry.

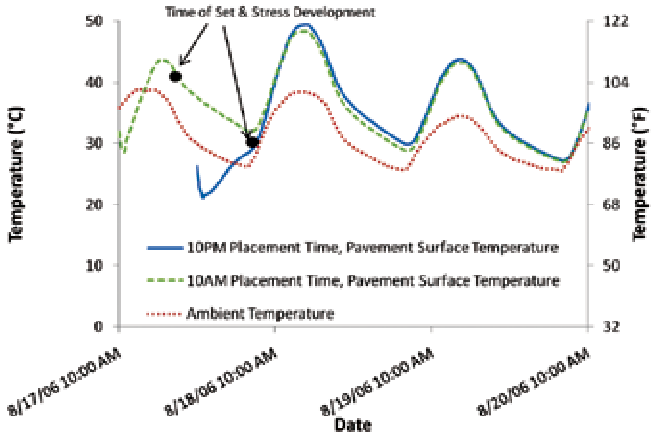

Fig. 8: Concrete temperature development for a 10 in. (254 mm) thick concrete pavement calculated using ConcreteWorks

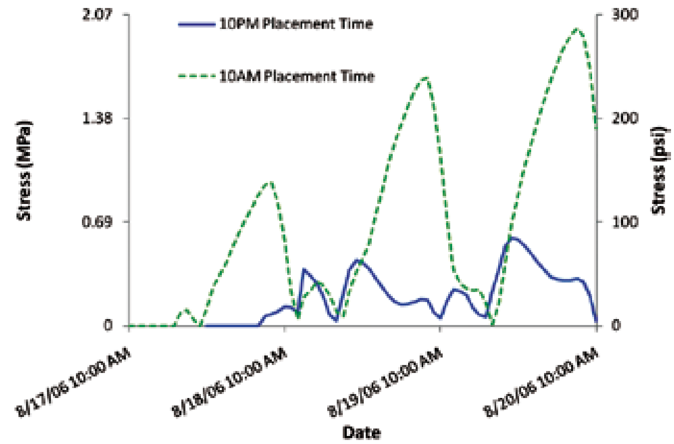

Fig. 9: Maximum tensile stress after concrete placement calculated for a 10 in. (254 mm) thick concrete pavement using HIPERPAV III

Construction practices

The final opportunity for the contractor to influence the temperature rise within a concrete element is via the construction practices employed. Numerous techniques are employed in practice including the use of chilled aggregates; using ice as a part of the mixing water; employing liquid nitrogen cooling of the concrete mixture at the job site[15]; and using cooling pipes, insulating blankets, or tenting with heaters to control the temperature development during the critical first few days. The weather can't be controlled, but through proper planning, weather extremes can be accommodated without sacrificing concrete performance.

SUMMARY

The nearly nonexistent supply of coarse cements in the U.S. requires engineered solutions to reduce the maximum temperature rise in concretes. Careful planning and testing is required to assure that concrete temperatures remain within acceptable limits through placement and curing. Contractors must be aware that cements have changed in the past 50 years, with a concurrent increase in the potential for generating higher internal temperatures.

Multiple mitigation strategies for minimizing undesirable temperature excursions are available, including materials selection, mixture proportioning, construction scheduling, and construction practices. Such an extensive toolbox should provide the knowledge and skills necessary to control concrete internal temperatures and thereby minimize problems of thermal cracking, reduced in-place (ultimate) strength development, and delayed ettringite formation.

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Note: Additional information on the ASTM Standards discussed in this article can be found at **www.astm.org**.

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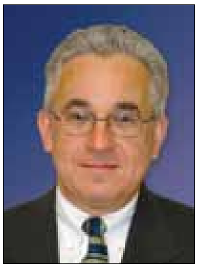

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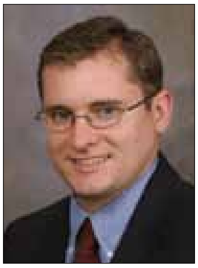

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