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Effect of Mixing and Placing in Hot Weather on Hardened Concrete Properties

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Abstract: Portland cement concrete exposed to high temperatures during mixing, transporting, casting, finishing, and curing can develop undesirable characteristics. Applicable requirements for such the hot weather concrete differ from country to country and government agencies. The current study is an attempt at evaluating the hardened properties of the concrete exposed to hot weather in fresh state. First of all, this study reviews the current state of understanding and practice for hot weather concrete placement in US and then roadway sites with suspected hot weather concrete problems were investigated. Core samples were obtained from the field locations and were analyzed by standard resonance frequency analysis and the boil test. Based on the results, there does not appear to be systematic evidence of frequent cracking problems related to high temperature placement. Thus, the suspicious deteriorations which are referable to hot weather concreting would be due to other factors.

Keywords: hot mix concrete, frequency resonance test, boil test, temperature limit, dynamic modulus, permeable pore space.

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 (Newman 1971; Soroka 1993; Turton 1995). 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 (RILEM 1981; Soudki et al. 2001). 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 (Samarai et al. 1983; 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 (Lee 1989; Schrader 1987). An important precaution is to ensure that the plastic concrete temperature be kept suitably below some defined threshold temperature (Malisch 1990). 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 (Bergin and Syed 2002). ACI Committee 305 (2006) 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 Committee 305 (2006) 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 Committee 305 2006).

Many researchers have tried to explain the adverse effects of the hot weather concreting on the hardened concrete properties, but there still exist various theories such as Feret's relation considering strength and design factors, Arrhenius law associated with strength and maturity and the hydration kinetics (Kayyali 1984; Mouret et al. 2003; Ortiz et al. 2005). Such researches have been conducted under well-controlled laboratory condition, so they cannot reflect the actual field condition with many variables. Furthermore, there is no clear evidence for the unfavorable effects under hot weather. Recently, some researches have reported unconventional results on the hot weather concreting. Mustafa and Yusof (1991) showed that the outdoor

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shrinkage under hot weather could be less than the controlled indoor shrinkage under same temperature condition and that the long-term effects of the hot weather might not be adverse as those usually reported in other researches. Ait-Aidera et al. (2007) found that the addition of water under hot weather can offer sufficient moisture to the hydration process to evolve under more or less valid conditions even though an increase in W/C ratio would generally lead to a fall in the concrete strength. In this context, it is worthwhile to investigate the effects of hot weather concreting on the hardened concrete properties under outdoor field condition because there is still no marked influence on the hardened properties.

The main objective of this study is to develop a better understanding of the effects of higher temperatures on hardened concrete properties of roadway concrete. This study first reviews the current state of understanding and practice for hot weather concrete placement in U.S. The three roadway sites with suspected hot weather concrete problems were investigated. Core samples were obtained from the field 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.

2. Surveys of State's Practice in US

To obtain information on states' practices for hot weather concreting in US, 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.

The results of the survey are shown in Table 1. 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 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).

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 5 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 Fig. 1. 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.

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 Fig. 2. From the survey results, it would seem that having no limit on concrete temperature may lead to increased likelihood of distressed concrete.

3. Field Site Visits

Three roadway sites with suspected hot weather concrete problems were investigated. This task required close assistance from the Illinois Department of Transportation (IDOT) Bureau of Materials and Physical Research (BMPR) and the district engineers.

The three locations visited were in Cruger road (Washington, IL), Driveway pavement off US-150 (Peoria, IL) and I-74 Ramp (Peoria, IL). This study includes major interstate highway pavement, 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.

3.1 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

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 states
100 °F	1 states

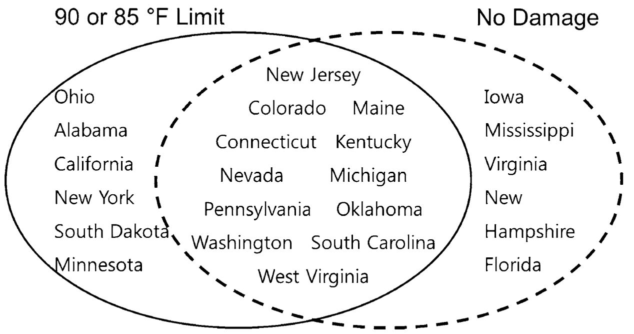

Fig. 1 Venn diagram representing the apparent trend between state that have a 90 °F (32.2 °C) or lower limit and report no deterioration.

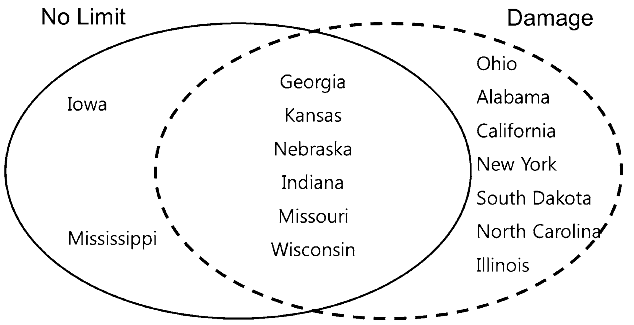

Fig. 2 Venn diagram representing the trend for states with no limit and pavement damage.

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 on March 6, 2008.

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 min. 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 (3.2–7.0 cm), 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 ~25 min. Concrete slump varied from 1.25 to 2.00 inches (3.2–5.1 cm), 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.

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 Fig. 3. Figure 4 shows a poor finished surface, and Fig. 5 shows an adequately finished surface for Cruger Road.

Fig. 3 Excessive joint spalling on the eastbound direction of Cruger Road, as well as poor surface texture.

Fig. 4 Eastbound Cruger Road, showing areas of poor surface friction, which are reflecting the sunlight.

3.2 Driveway Pavement off US-150, Peoria, IL

This pavement was a small piece of driveway pavement that was an example of poor summer paving technique. This driveway was poured in August 2002, starting at about 4:00 in the afternoon. The load of concrete was one previously rejected from a nearby poor because of excess time since batching, about 90 min. After the truck was rejected, it returned within 45 min, 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 stuck off, it began setting and the finishers had a very difficult time and brooming the surface.

These concrete slabs have almost no surface texture and significant pop-out 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

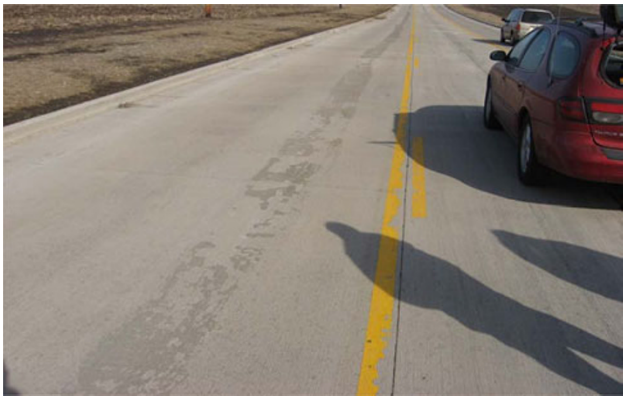

Fig. 5 Westbound Cruger Road with proper surface texture (*left lanes*). The discoloration in the westbound lane is merely a cosmetic flaw.

Fig. 6 No surface texture and significant pop-out distress.

freezing damage during the winter. This damage can be seen in Fig. 6.

3.3 I-74 Ramp, Peoria, IL

This section, an off-ramp from interstate 74, was visited on March 6, 2008. The pour on the ramp started at about noon on June 25, 2005. According to 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 truck to wait 45–60 min 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 places, the pour went better.

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.

Fig. 7 Crazing and poor tining on exit ramp jointed concrete pavement.

Though this is not a problem yet, it could be 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 Fig. 7.

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.

4. Experimental Test on Field Samples

Experimental tests were carried out in order to better understand the effects of high placement temperature on concrete. Core samples were obtained from the three different locations. 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 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. However, the common strength tests such as the compressive and flexural strength tests were not conducted because considerable small and micro-cracks were distributed on the top and near surfaces of the core samples and the top and bottom surfaces were not proper for the capping.

The resonance frequency test (ASTM C215 2008) 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 C642 2006) determines the bulk density, absorption, and the volume percentage of permeable pore space within the concrete sample.

Table 2 Summary of the core samples.

Road/location	Pour	Distress	Plastic concrete temperature	Core label
Cruger Road	WB	None	94 °F	CRWB1 CRWB2
	EB	Poor finishing, crazing	91 °F	CREB1 CREB2
US150	Driveway	Poor finishing, popouts	Unknown	150D1 150D2
	Ramp/gutter	None	67 °F	150R1 150R2
I-74	Ramp	Poor finishing, crazing	93 °F	74R1 74R2 74R3
	Shoulder	None	60° F	74S1 74S2

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.

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.

4.1 Resonance Frequency Test

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.

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 5 cm long samples.

The resonant frequency testing procedure is outlined in ASTM C215 (2008), 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.

4.2 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 C642 2006). 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.

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 h, then allowed to cool in the water for at least 14 h. 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.

5. Results of Experimental Tests

5.1 Results of Resonance Frequency Analysis

Table 3 shows the dynamic Young's moduli obtained with the resonance test for 10 cm core samples. A statistical analysis of the results follows. 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. It can be estimated if two sample

Table 3 Dynamic modulus results for 10 cm core lengths.

Core label	Hot/cold	Short label	Mass (g)	Diameter (cm)	Aggregate type	Modulus (Gpa)
US 150 Driveway #1	Hot	150D	1824	10.03	Limestone	31.57
US 150 Driveway #2	Hot	150D2	1762	10.03	Limestone	27.99
US 150 Ramp #1	Cold	150R1	2030	10.03	River rock	36.00
US 150 Ramp #2	Cold	150R2	2009	10.03	River rock	35.25
I-74 Ramp A-1 #1	Hot	74R1	1805	10.03	Limestone	31.13
I-74 Ramp A-1 #2	Hot	74R2	1516	10.03	Limestone	31.06
I-74 Ramp A-1 #3	Hot	74R3	1788	10.03	Limestone	32.49
I-74 Shoulder #1	Cold	74S1	1733	10.03	Limestone	30.44
I-74 Shoulder #2	Cold	74S2	1817	10.03	Limestone	33.09
Cruger Road EB #1	Hot	CREB1	1914	10.03	River rock	36.80
Cruger Road EB #1	Hot	CREB2	1902	10.03	River rock	39.96
Cruger Road WB #2	Hot	CRWB1	1947	10.03	River rock	39.98
Cruger Road WB #2	Hot	CRWB2	1934	10.03	River rock	42.12

populations are distinct from each other, to a certain level of statistical confidence, using the T test knowing the sample mean and variance of our test samples, for a specific number of samples. For a two-tailed test, the degree of statistical confidence in percent is determined by $100(1 - 2 \times \alpha)$, where α is the area under the T distribution curve above a computed T value. The T value is computed as follows:

$$T = (X_1 - X_2)/[S_P(1/n_1 + 1/n_2)^{0.5}] \quad (1)$$

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.

Limit values of T are established for a given α value and number of samples. In this study, a 95 % confidence level is assumed, meaning that $\alpha = 0.025$. If the computed T is greater than the limit value of T for $\alpha = 0.025$ 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 resonance of 10 cm core samples, with regard to aggregate type and temperature condition.

Groups	Count	Mean (GPa)	Variance (GPa)
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

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

Groups	Count	Mean (GPa)	Variance (GPa)
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

Table 6 Dynamic modulus results for 5 cm core lengths.

Core label	Hot/cold	Short label	Mass (g)	Aggregate type	Modulus (Gpa)
US 150 Driveway #1	Hot	150D1-TOP	891.62	Limestone	27.6
US 150 Driveway #1	Hot	150D1-MID	885.61	Limestone	28.32
US 150 Driveway #2	Hot	150D2-TOP	841.2	Limestone	25.63
US 150 Driveway #2	Hot	150D2-MID	875.5	Limestone	28.55
US 150 Ramp #1	Cold	150R1-TOP	1042.71	River rock	39.64
US 150 Ramp #1	Cold	150R1-MID	937.46	River rock	28.98
US 150 Ramp #2	Cold	150R2-TOP	1077.69	River rock	38.78
US 150 Ramp #2	Cold	150R2-MID	881.15	River rock	29.26
I-74 Ramp A-1 #1	Hot	74R1-TOP	967.7	Limestone	36.95
I-74 Ramp A-1 #1	Hot	74R1-MID	789.67	Limestone	25.79
I-74 Ramp A-1 #2	Hot	74R2-TOP	940.02	Limestone	38.11
I-74 Ramp A-1 #2*	Hot	n/a	No data	No data	No data
I-74 Ramp A-1 #3	Hot	74R3-TOP	874.42	Limestone	31.2
I-74 Ramp A-1 #3	Hot	74R3-MID	864.16	Limestone	30.04
I-74 Shoulder #1	Cold	74S1-TOP	884.22	Limestone	29.75
I-74 Shoulder #1	Cold	74S1-MID	800.33	Limestone	26.7
I-74 Shoulder #2	Cold	74S2-TOP	914.81	Limestone	30.35
I-74 Shoulder #2	Cold	74S2-MID	854.11	Limestone	30.08
Cruger Road EB #1	Hot	CREB1-TOP	999.05	River rock	41.22
Cruger Road EB #1	Hot	CREB1-MID	866.01	River rock	34.58
Cruger Road EB #2	Hot	CREB2-TOP	977	River rock	33.46
Cruger Road EB #2	Hot	CREB2-MID	873.82	River rock	25.68
Cruger Road WB #1	Hot	CRWB1-TOP	958.18	River rock	33.12
Cruger Road WB #1	Hot	CRWB1-MID	938.33	River rock	32.05
Cruger Road WB #2	Hot	CRWB2-TOP	997.91	River rock	35.34
Cruger Road WB #2	Hot	CRWB2-MID	884.1	River rock	34.74

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.

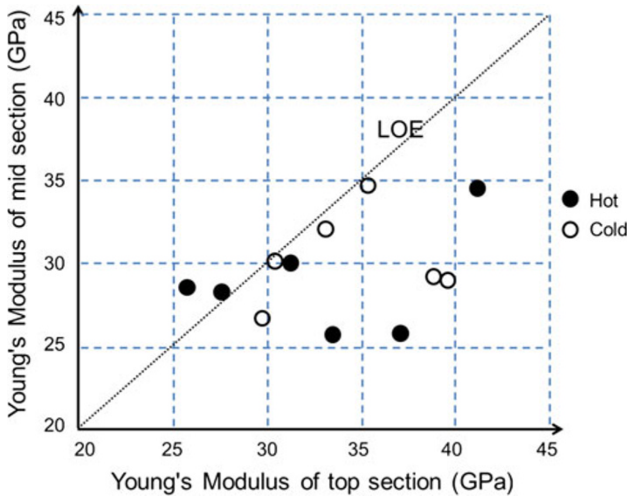

Fig. 8 Comparison of dynamic Young's modulus from top and mid sections of same cores: comparison with regard to plastic temperature across all aggregate types.

Table 6 shows the results for the 5 cm 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 8 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 LOE.

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.

5.2 Results of Density, Absorption, and Void Content Tests

Table 7 shows the results from the boil test for the core sections followed by a statistical analysis of the obtained data.

The data were analyzed using the same statistical T test, described before. As before, a 95 % confidence level is assumed. 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 8. A first look at the data in Table 8 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.

6. Summary of Tests on Field Samples

Laboratory tests that are sensitive to distributed damage content and permeable void volume 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, it is pointed 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 it is concluded that the distributed damage state and permeable pore structure of concrete is not significantly affected by concrete plastic temperatures within this sample

Table 7 Boil test results for 5 cm core lengths.

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	2.3	2.42	2.42	2.63	12.60
150R1-MID	5.18	5.35	2.3	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	2.4	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	2.4	2.41	2.6	12.04
CREB1-MID	5.42	5.66	2.26	2.39	2.39	2.6	12.81
CREB2-TOP	4.97	5.23	2.3	2.41	2.42	2.61	12.01
CREB2-MID	5.23	5.47	2.27	2.39	2.4	2.6	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

set, although there is evidence of some moderate effects. However, considering the well-known problems caused by hot weather concreting and the results provided by this study, the authors suggest further testing to understand the behavior of concrete at high placement temperatures.

7. Conclusion

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

- 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.
- The concrete at the three sites with suspected hot weather problems 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 is not clear.
- Based on a 95 % 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 %.

Table 8 Statistical results of permeable pore space values obtained resonance of 5 cm core samples, with regard to aggregate type and temperature condition, and core section.

Groups	Count	Mean (%)	Variance (%)
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

- Based on these data from the test site samples, it is concluded that the distributed damage state and permeable pore structure of concrete is not significantly affected by concrete plastic temperatures within this sample set.
- It may be concluded that the suspicious deteriorations and cracks which are referable to hot weather would be due to other factors.

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