EFFECTS OF COOLING RATES ON THE DURABILITY OF CONCRETE

Chung-Hsing Lin, Carolina Power and Light Company; and
Richard D. Walker and W. W. Payne, Department of Civil Engineering,
Virginia Polytechnic Institute and State University

Past research generally assumed that rapid cooling rates cause faster deterioration of concrete that is susceptible to damage from freezing and thawing. The objective of this project was to investigate the effect of varying freezing rates on an otherwise standard ASTM test. Eighty-one concrete specimens were fabricated with an aggregate capable of causing deterioration under freezing-and-thawing conditions. The aggregates were placed in the concrete at 3 different degrees of saturation. Three rates of cooling were used: 4.4 F/hour (2.45 C/hour); 6.6 F/hour (3.67 C/hour); and 13.3 F/hour (7.39 C/hour). Modifications to freezing and thawing equipment are described, and possible explanations of the results obtained are presented. If the aggregate was not initially saturated when placed in the concrete, slower freezing rates produced demonstrably faster rates of deterioration. It is theorized that a slower rate of cooling enables more water to migrate to the surroundings of the coarse aggregate. Therefore, during the thawing phase, more water is available for the coarse aggregate to become increasingly saturated. Rate of cooling seemed not to affect the rate of deterioration of concrete containing aggregates placed in the concrete already in the saturated state.

•AS one of the most abundant construction materials in the world, concrete by the thousands of tons is used to build a variety of structures exposed to natural weathering forces. Methods of testing the resistance of concrete materials to these weathering forces have been studied since at least the late 1800s. In the temperate zones, alternate freezing and thawing are considered among the most destructive of the natural weathering conditions.

Many organizations have worked with various methods of evaluating concrete in the laboratory for the purpose of predicting durability in the field, especially since the 1940s. ASTM, beginning in 1952 and 1953, originally described 4 procedures (essentially, slow and fast procedures for freezing in water and thawing in water and slow and fast procedures for freezing in air and thawing in water—C290-52T, C291-52T, C292-52T, and C310-53T). In a cooperative program involving 13 laboratories, an attempt was made to compare these tests (1). Aside from showing that the freezing-in-water tests caused concrete to deteriorate more quickly than those that freeze in air, the tests demonstrated that the various laboratories had difficulty in obtaining consistent results. All methods provided means of distinguishing between known good concrete and bad, with the rapid freezing-in-water procedure being the quickest and most consistent. The slow freezing-in-water tests gave essentially the same results but required more time. In general, both the slow and fast tests use about the same rate of cooling; the fast test has more cycles in a 24-hour period.

ASTM currently describes 3 procedures, 2 of which are in C666-73, "Standard Method of Test for Resistance of Concrete to Rapid Freezing and Thawing" and rep-

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resent a combination of the old C 290 and C 291, the fast freezing-in-water and fast freezing-in-air procedures. A third method, C 671-72T, "Tentative Method of Test for Critical Dilation of Concrete Specimens Subjected to Freezing", evolved from suggestions made by Powers (2, p. 1150) and later developed by the California Department of Highways (3) and Larson and Cady (4). This test restricts the rate of cooling to 5 ± 1 F (2.775 ± 0.556 C). This latter test, however, is not in common use. Tests using the fastest rates of cooling are similar to that used by the Corps of Engineers (2, p. 1148), which has cooling rates up to 36 F/hour (20 C/hour).

Except for C 671, criticism of freezing and thawing procedures in general has often centered around the fact that the rates of cooling used in the laboratory tests are much greater than those found in the field, thus reducing the possibility of good correlation between laboratory results and field performance. T. F. Willis (5, p. 1140) summarized this view by stating:

> The fastest rate of cooling to which pavements in this country are subjected under service conditions is 6 F (3.336 C) per hour. If rate of cooling is a factor in the deterioration of concrete exposed to frost action, the acceleration achieved by these test procedures is somewhat analogous to accelerating a test of a glass shelf, intended as a support for a light flower pot, by hitting it with a sledge hammer.

This paper compares laboratory test results of an air-entrained concrete (containing an aggregate generally recognized as "poor" and placed in the concrete at various levels of saturation) exposed to different rates of cooling in alternate cycles of freezing and thawing.

RATE OF COOLING

Properly made air-entrained concrete containing "good" aggregate will withstand many alternate cycles (up to 1,000 in some tests of laboratory freezing and thawing) without exhibiting significant deterioration. An exception to this is concrete tested by procedures where extremely rapid freezing is obtained by circulating brine at -20 F (-28.9 C) around a specimen in a rubber boot filled with water. However, most laboratory procedures use 2 or more hours to lower the temperature of the specimen from 40 F (4.4 C) to 0 F (-17.8 C), with cooling rates between 6 F (3.33 C) and 20 F (11.11 C) per hour.

It is thought that the more rapid the cooling rate, the more the specimen suffers thermal shock, which possibly masks the effects of aggregate susceptibility. As indicated by Powers' hypothesis (6), the faster the cooling, the greater the velocity of movement of unfrozen water and the greater the stresses induced. In addition, the rate of concrete contraction is more rapid than the water expansion. If the specimen is in a dry condition, the rate of cooling should have little effect on durability. If a specimen is saturated, a maximum stress condition will occur.

When thawing water is circulated, a second shock occurs. The rapid rate of thawing used with most laboratory methods [0 F (-17.8 C) to 40 F (4.4 C) in ½ to 1 hour in most tests] will cause rapid movement of unfrozen water, but the concrete will be expanding and the water contracting, thus eliminating some of the stress encountered in rapid cooling (6).

Because the rate of thawing is less likely to be significant, and because of the limitations of equipment available for this study, only the rate of cooling is considered here.

OBJECTIVE

The objective of this project was to extend knowledge of concrete durability by investigating the effect of different freezing rates. To achieve this objective, a nondurable aggregate was subjected to 1 of 3 moisture conditions: (a) air-dried; (b) 24-hour water soaking; and (c) 24-hour vacuum saturation at 1.5 cm mercury. The aggregates were incorporated in concrete specimens 3 × 3 × 16 in. (7.62 × 7.62 × 40.64 cm), fabricated with a carefully controlled air content, and subjected to alternate cycles of freezing and thawing between 40 F (4.4 C) and 0 F (-17.8 C). Three average rates of cooling were used: 4.4 F/hour (2.45 C/hour), 6.6 F/hour (3.67 C/hour), and 13.3 F/hour (7.39 C/hour).

SCOPE AND DESCRIPTION OF WORK

Eighty-one specimens were used to investigate the effects of various rates of cooling on the frost resistance of concrete made with "poor" aggregates placed in the concrete at different levels of saturation. For each of the 3 rates of cooling, 9 specimens were made with aggregates saturated at the 3 different levels of moisture. For each mix design, 3 specimens were made on different dates. All concrete was air-entrained, with air contents averaging 6 percent. The concrete was designed for a slump of 3 in. (7.62 cm) and a compressive strength of 3,500 psi (24 132 kPa).

Procedures

The 81 specimens were exposed to alternate cycles of freezing and thawing after 13 days of curing in lime-saturated water. ASTM C 666-71 was followed for fast freezing and thawing in water, using the Logan apparatus developed by Cordon (7). The specimens were measured for length, weight, and dynamic modulus before beginning the test and every 6 to 10 cycles thereafter. Also, length-change measurements were made at intervals of approximately 2 F (1.112 C) during the initial freezing and thawing cycle. Testing was continued until the specimen had lost 40 percent of its original dynamic modulus or had undergone 100 freezing cycles.

Materials

The "poor" aggregate was the float material (at specific gravity of 2.55) from a heavy media plant in the midwest using a glacial gravel source. Although heterogeneous from a mineralogical standpoint, the amount of weathered chert was such that a 100-cycle durability factor of about 5 was obtained when the aggregate was used in vacuum-saturated condition. It is from the same material source described in full as aggregate H in NCHRP Report 12 (8).

A single-source type III cement was used throughout the study, and Vinsol resin added at the mixer was used to control the air content.

A local fine aggregate of crushed limestone sand was used in all specimens. The sand has a fineness modulus of 2.75, an absorption of 1.07 percent, and a bulk specific gravity of 2.60.

Equipment Modification

The freeze-thaw equipment in the Virginia Tech concrete laboratory was not designed to be regulated for various rates of cooling. To vary rates of cooling for this study, a micro-adjusting valve was installed in the system, as shown in Figure 1. When a slow rate of cooling was desired, the valve was regulated to a small opening, allowing a small amount of Freon liquid to be pumped into the evaporator. On the other hand, if a rapid rate of freezing was used, the valve was opened wider, allowing a larger amount of Freon liquid to be pumped into the evaporator.

RESULTS AND DISCUSSION

Complete results are given in Tables 1, 2, and 3, with the averages shown representing the values discussed. These tables also serve to demonstrate the kind of variability that existed in the test results as well as to provide information difficult to present in graphical form. Mix V refers to specimens made with vacuum-saturated aggregate, mix S to specimens with aggregate soaked for 24 hours, and mix A to specimens made with aggregate that was air-dry (except for a brief wetting period) when put into the concrete.

Statistical Analysis

Simple statistical analysis (analysis of variance) substantiates what is apparent merely from viewing the data. For mix V (aggregates vacuum-saturated), the analyses of the test results affirm the hypothesis that the concretes, when subjected to alternate cycles of freezing and thawing, have the same durability under various rates of cooling. Concerning concretes made with soaked and air-dried aggregates, the analyses reject this hypothesis; thus, it is concluded that these concretes, when subjected to alternate cycles of freezing and thawing, are affected by the cooling rate. Figure 2 shows this conclusion.

Significance and Hypothesis of Failure

At first glance, the surprising result is that slower cooling rates apparently produce loss of strength in the concrete more quickly than the fastest rate, especially where the aggregate is not vacuum-saturated. Actually, the concrete containing the vacuum-saturated aggregate was so low in durability that no difference was displayed.

Considering that the literature implies that faster cooling rates should produce more rapid destruction, the results from the other 2 mixes are indeed interesting. It is easy to surmise that the time needed for aggregate in the S and A mixes to become critically saturated is an important part of the answer to the question of why the slower rate of cooling results in lower concrete durability.

Time required for critical saturation of an aggregate is discussed by Verbeck and Landgren (9), who point out that, after a significant drying period, some aggregates may need only a few weeks of wetting to become critically saturated. In these tests, all specimens are placed in water for 2 weeks before placing into test, a test that keeps the specimens wet. The results indicate, however, that even after this test period the aggregates in the S and A mixes are not critically saturated. Since failure occurs at a maximum of about 60 cycles, only 10 to 14 more days are required for failure and presumed critical saturation. It would appear that the freezing process greatly accelerates the saturation process.

If freezing accelerates the saturation process, what are the mechanisms involved? The Verbeck and Landgren paper (9) and the works by Powers (2, 6) discuss many aspects of this. It is not believed that the limited data in this paper refute statements in those works. On the other hand, those articles do not present explanations that are completely applicable to the test conditions to which the S and A mixes in this study were exposed.

It has been recalled to one of the authors that Stanton Walker (then Director of Engineering at the National Ready-Mixed Concrete Association) argued on the floor of some forum that the fact that the slow freezing of turkeys is detrimental to the quality of the meat had nothing whatever to do with the freezing and thawing of concrete. We are inclined to agree with that statement but not with the implication that fast freezing is more detrimental than slow with regard to concrete.

Nerenst (10) in 1960 indicated that the boundary layer between aggregate and paste is a locus for conditions favorable for the formation of ice until a certain degree of hydration of the surrounding paste is reached. He also hypothesized that, as paste with high amounts of water surrounds coarse aggregate with low porosity, the heat

Figure 1. Freezing and thawing system.

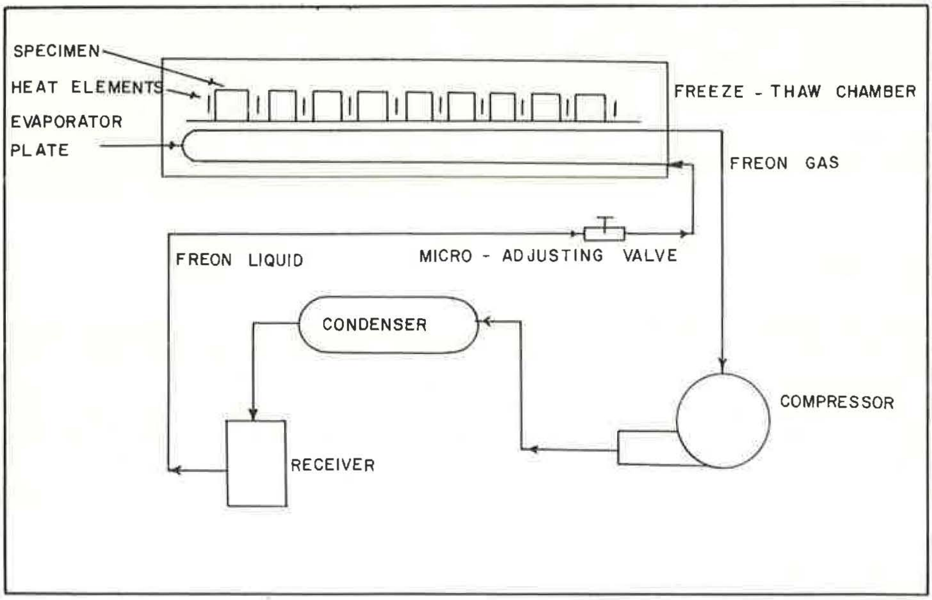


Table 1. Cooling rates versus concrete durability of mix V.

Cooling Rate	No.	Relative Dynamic Modulus (percent) 90 C	90 L	80 C	80 L	70 C	70 L	60 C	60 L	DF100
F	1	1.74	17.4	3.48	35.0	5.19	62.0	8.0	89.0	4.80
	2	3.18	26.0	4.92	27.0	6.39	34.0	7.5	45.0	4.50
	3	1.37	19.0	2.74	39.0	4.57	54.0	5.5	72.0	3.30
	4	1.35	26.0	2.70	36.0	3.35	49.0	4.0	55.0	2.40
	5	1.51	8.0	3.02	13.0	4.00	17.0	5.0	21.0	3.00
	6	1.00	17.0	2.00	33.0	3.00	42.0	4.0	50.0	2.40
	7	1.40	13.0	2.80	25.0	6.00	59.0	8.1	86.0	4.86
	8	1.86	19.0	3.72	33.0	6.00	53.0	7.5	76.0	4.50
	9	1.27	21.0	2.54	42.0	3.81	55.0	5.0	74.0	3.00
	Average	1.63	15.6	3.10	31.4	4.70	47.2	6.1	63.1	3.64
M	1	3.30	24.0	5.00	31.0	6.46	40.5	8.0	49.0	4.80
	2	1.56	13.0	4.67	34.0	5.80	47.0	7.0	60.0	4.20
	3	2.16	23.3	4.16	40.0	5.24	44.8	6.3	50.0	3.80
	4	3.58	29.2	6.00	44.0	7.20	56.0	8.2	68.0	4.90
	5	2.38	17.8	4.53	32.8	6.18	42.0	7.5	52.0	4.50
	6	2.35	21.2	4.54	40.0	6.36	55.0	8.0	68.0	4.80
	7	1.33	14.8	2.66	29.6	4.00	46.0	5.0	68.0	3.00
	8	1.10	17.0	2.00	34.0	3.10	42.0	4.0	50.0	2.40
	9	0.98	12.0	1.92	20.5	2.84	35.3	4.2	51.0	2.50
	Average	2.10	19.1	3.94	34.0	5.20	45.4	6.5	57.3	3.88
S	1	1.62	9.7	3.24	19.4	4.86	29.2	6.3	42.0	3.80
	2	1.41	16.1	2.82	32.2	4.23	48.2	5.9	70.0	3.50
	3	1.37	12.3	2.74	24.6	4.11	37.0	5.7	50.0	3.40
	4	1.69	16.9	3.38	33.8	5.07	50.7	6.5	64.0	3.40
	5	1.90	15.2	3.80	30.4	5.70	45.6	8.0	68.0	4.80
	6	1.62	16.2	3.24	32.4	4.86	49.0	6.5	64.0	3.90
	7	1.11	28.8	2.22	57.6	3.33	85.6	4.5	117.0	2.70
	8	0.02	10.1	1.84	20.2	2.76	30.3	4.0	50.0	2.40
	9	1.35	13.6	2.70	32.2	4.05	48.4	5.5	60.0	3.30
	Average	1.44	15.4	2.89	31.4	4.33	47.1	5.9	65.0	3.52

Note: C = cycle, L = cumulative length change. F = 13.3 F/hour, M = 6.6 F/hour, S = 4.4 F/hour (1 F/hour = 0.556 C/hour).

Table 2. Cooling rates versus concrete durability of mix S.

Cooling Rate	No.	Relative Dynamic Modulus (percent) 90 C	90 L	80 C	80 L	70 C	70 L	60 C	60 L	DF100
F	1	11.4	14.5	23.6	30.0	32.0	38.0	44.0	61.0	26.4
	2	11.1	27.0	27.0	44.0	35.3	58.4	44.0	69.0	26.4
	3	11.2	26.0	22.0	21.0	32.0	27.0	44.0	42.0	26.4
	4	6.0	22.0	9.5	39.0	12.5	53.0	16.0	76.0	9.6
	5	7.0	26.0	12.5	31.0	18.2	44.0	33.0	68.0	19.8
	6	10.1	24.0	20.5	29.0	39.0	58.0	50.0	86.0	30.0
	7	7.5	19.0	13.1	36.0	21.0	64.0	29.0	91.0	17.4
	8	5.0	13.0	14.6	35.5	26.0	58.0	32.0	79.0	19.2
	9	12.5	22.0	27.6	50.0	33.0	62.0	45.0	93.0	27.0
	Average	9.1	21.5	18.9	35.1	27.7	51.4	37.4	73.9	22.5
M	1	7.0	16.0	16.0	27.0	21.0	39.0	25.0	49.0	15.0
	2	5.0	20.0	13.0	30.0	24.8	49.0	26.0	55.0	15.6
	3	9.0	13.0	24.0	40.0	28.4	43.0	34.0	68.0	20.4
	4	5.5	12.0	13.0	17.0	17.0	28.0	20.0	45.0	12.2
	5	6.0	15.0	25.4	25.0	30.0	33.0	44.0	60.0	26.4
	6	7.7	29.0	26.0	47.0	29.0	54.0	34.0	66.0	20.4
	7	9.6	16.0	15.5	22.0	22.0	35.0	33.0	58.0	19.8
	8	8.2	18.4	12.0	28.3	21.2	38.3	29.0	54.6	17.4
	9	6.0	16.2	13.0	31.1	24.3	43.4	32.7	60.0	19.6
	Average	7.1	17.3	17.5	29.7	24.6	40.9	30.8	57.3	18.5
S	1	6.0	16.0	11.5	33.0	14.5	46.0	17.0	58.0	10.2
	2	6.0	11.0	10.0	20.0	14.0	42.0	18.5	65.0	11.1
	3	10.0	23.0	20.1	34.0	24.0	40.0	27.0	50.0	16.2
	4	3.5	16.0	7.0	31.0	11.0	48.0	14.5	64.0	8.7
	5	8.6	13.5	16.0	21.0	21.0	37.0	23.6	45.0	14.2
	6	8.6	11.5	15.5	20.0	21.0	25.0	25.0	34.0	15.0
	7	4.0	19.0	8.0	33.0	11.0	40.0	14.0	50.0	8.4
	8	4.5	14.0	9.0	26.0	12.0	36.0	16.0	45.0	9.6
	9	7.8	10.0	16.0	24.0	21.0	34.0	24.0	43.0	14.4
	Average	6.6	14.9	12.6	26.9	16.6	38.7	20.0	50.4	12.0

Note: C = cycle, L = cumulative length change. F = 13.3 F/hour, M = 6.6 F/hour, S = 4.4 F/hour (1 F/hour = 0.556 C/hour).

Table 3. Cooling rates versus concrete durability of mix A.

Cooling Rate	No.	Relative Dynamic Modulus (percent) 90 C	90 L	80 C	80 L	70 C	70 L	60 C	60 L	DF100
F	1	5.0	13.0	10.0	26.0	26.0	39.0	47.0	67.0	28.2
	2	11.0	11.5	20.0	15.0	33.5	28.0	46.0	46.0	27.6
	3	10.0	21.0	43.4	35.0	58.0	38.0	72.0	40.0	43.2
	4	31.0	17.0	55.0	34.0	66.0	47.0	79.0	57.0	57.4
	5	32.0	17.0	55.0	52.0	67.0	78.0	79.0	127.0	57.4
	6	24.0	29.0	39.0	39.0	59.0	72.0	67.0	88.0	40.2
	7	8.0	10.0	25.3	25.3	42.0	44.0	56.0	58.0	33.6
	8	10.0	9.0	42.0	42.0	54.5	28.0	62.0	29.0	37.2
	9	8.2	18.0	23.2	23.2	35.4	49.0	50.0	90.0	30.0
	Average	15.5	17.1	34.8	30.6	49.0	47.0	62.0	66.8	39.4
M	1	13.0	7.6	16.0	19.0	32.7	51.0	48.0	83.0	28.8
	2	6.7	10.8	24.8	31.6	36.0	48.0	40.0	50.0	24.0
	3	14.0	8.5	29.5	23.0	49.0	37.0	57.0	47.0	34.2
	4	15.0	18.0	30.0	36.0	44.0	51.0	54.0	60.0	32.4
	5	16.0	11.7	28.0	19.0	36.0	26.0	52.0	48.0	31.2
	6	8.9	18.8	17.8	37.6	27.0	51.0	42.0	70.0	25.2
	7	8.1	11.1	29.0	37.0	41.0	55.0	50.0	85.0	30.0
	8	34.0	6.0	50.0	21.0	62.0	30.0	76.0	40.0	45.6
	9	22.0	6.0	37.6	21.0	42.8	34.0	54.0	48.0	32.4
	Average	15.3	10.9	29.2	27.2	41.2	42.6	52.6	59.0	31.5
S	1	15.1	16.0	28.7	23.0	44.0	42.0	46.5	45.0	27.9
	2	25.0	16.0	43.0	42.0	50.0	67.0	60.0	76.0	36.0
	3	12.0	5.0	22.0	5.0	28.0	8.0	8.0	11.0	20.7
	4	9.0	16.0	22.0	40.0	30.0	51.0	37.0	65.0	22.2
	5	8.4	6.0	18.7	32.0	30.0	42.0	37.5	60.0	22.5
	6	24.0	17.0	38.0	40.0	46.0	60.0	50.0	80.0	30.0
	7	7.0	23.0	17.0	32.0	27.0	51.0	35.0	66.0	21.0
	8	11.5	21.0	32.0	38.0	42.3	52.0	50.0	70.0	30.0
	9	26.0	25.0	45.8	48.0	52.0	53.0	55.0	55.0	33.0
	Average	15.3	16.1	29.7	33.3	38.8	47.3	45.1	57.5	27.0

Note: C = cycle, L = cumulative length change. F = 13.3 F/hour, M = 6.6 F/hour, S = 4.4 F/hour (1 F/hour = 0.556 C/hour).

Figure 2. Durability factors versus cooling rates for mix V, mix S, and mix A.

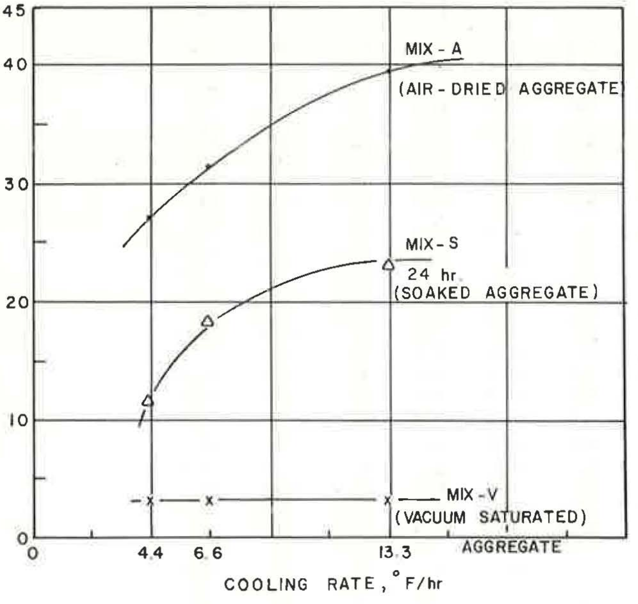
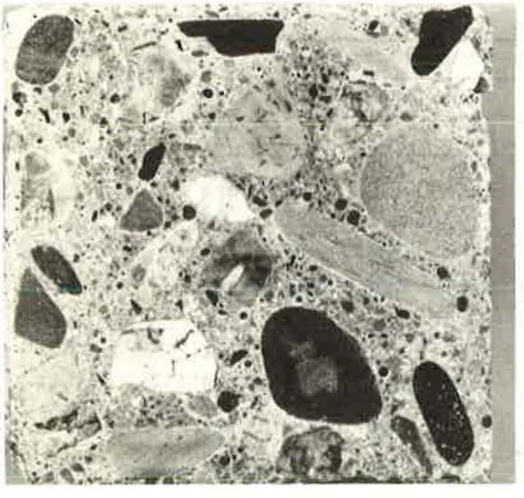


Figure 3. Section of concrete specimen after reaching 60 percent relative dynamic modulus of its original value.



released by the fusion of ice may delay the penetration of frost in the paste in comparison with the penetration of freezing temperatures into the aggregate. Hence, coarse aggregate may serve as centers of low temperature that extract water from the unfrozen paste in all directions.

Experiments by Khakimov (11) relating to soils demonstrated that during frost action the part played by water migration toward the freezing front decreases with increasing rate of cooling. It appears that during the freezing of concrete a slower rate of cooling allows more water to migrate to the surroundings of the coarse aggregate; therefore, during the thawing phase, more water is available with which the coarse aggregate may become increasingly saturated. Put another way, the faster the rate of cooling, the less water that moves to the boundary between aggregate and paste; thus it takes the aggregate much longer to obtain a critical saturation, resulting in greater concrete durability.

Figure 3 shows a section of the concrete specimen that lost 40 percent of its original dynamic modulus.

SUMMARY

From the foregoing discussion, the following statements might be made:

1. For concretes made with vacuum-saturated aggregates that are extremely poor in frost resistance, the slow rate of cooling tends to result in durability factors that are about equal to the low durability factors produced by the fast rate of cooling.
2. For concretes made with soaked or air-dried aggregates, the faster rate of cooling tends to result in greater concrete durability.
3. The slower the cooling rate, the more quickly the aggregates become critically saturated.

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DISCUSSION

Hormoz Famili, Azarabadegan University, Tabriz, Iran

The authors have made an interesting study of a fundamental parameter in freeze-thaw durability of concrete. The importance of rate of cooling has been emphasized in most of the reported research, and yet little experimental data have been presented to demonstrate the role of this factor quantitatively.

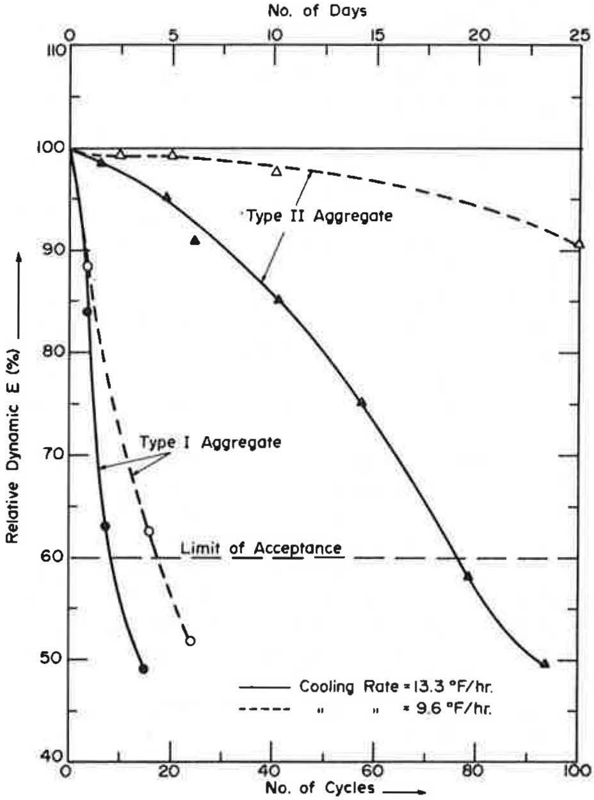
Although the limited data presented by the authors preclude a generalized conclusion, it has been shown that under a certain testing condition with a particular type of aggregate, contrary to the generally accepted view, slower rates of cooling could be more damaging to concrete than fast rates.

To demonstrate the importance of the testing conditions and the influence of various characteristics of aggregates, the results of tests performed by the writer are shown in Figure 4. In these tests aggregates were soaked in water for 24 hours prior to mixing (corresponding to mix S of the authors), and 24 hours after casting the concrete the specimens were placed in lime-saturated water for 13 days prior to commencement of the freeze-thaw tests. The freeze-thaw test procedure was a modified version of the ASTM C 666-71 test. The freeze-thaw cycle consisted of lowering the specimen temperature in humid air from 40 F to the minimum cooling temperature (0 F in fast cooling and 11 F in slow cooling) in 3 hours. The rates of cooling thus produced were 13.3 F/hour or 9.6 F/hour respectively. The limitation of the apparatus was such that the length of the cooling period could not be altered. At the end of the cooling period the specimen temperature was raised from the minimum temperature to 40 F gradually in 1 hour by means of humid air, and then water at a temperature of 40 F was sprayed on the specimen for 2 hours. In this manner the thermal shock usually experienced by the concrete at the end of the cooling period was avoided. Two types of aggregates were used. Type 1, which had a poor service record, was a mixture of sandstone and shale with a specific gravity of 2.46 and absorption value of 3.52 percent. Type 2 aggregate was a weathered flint gravel with the corresponding values of 2.54 and 2.05 percent respectively.

Figure 4 shows that increasing the rate of cooling from 9.6 F/hour to 13.3 F/hour reduced the durability factor from 10.8 to 4.8 for type 1 aggregate and from 92 to 46.8 for type 2 aggregate.

In conclusion, these results clearly illustrate the point that there are other significant factors, besides the rate of cooling, that affect the freeze-thaw behavior of aggregates in concrete. In one situation these factors could add up and produce a satisfactory performance for an aggregate, and in another situation they could cancel each other and result in an entirely unsatisfactory performance. The disparity in results reported by the authors and those mentioned here is most likely due to the empirical nature of the testing techniques used. Further fundamental research is needed to determine the influence of different parameters, individually and collectively, before the

Figure 4. Effect of rate of freezing on the freeze-thaw performance of air-entrained concrete with different types of aggregates.



mechanism(s) involved in the deterioration of concrete due to frost-susceptible aggregates can be adequately explained. Unless this important step is taken, these types of contradictory results, which have been frequently reported during the past 30 years, are to be expected, and little use can be derived from these results or these testing procedures for the quantitative indication of frost-susceptible aggregates, for which there is a great demand.

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AUTHORS' CLOSURE

The authors sincerely thank Famili for his pertinent and timely discussion of such an important phenomenon in the freeze-thaw durability of concrete.

In a study by Pence (13) there are recorded data indicating that for some aggregate particles there is a contribution to the freeze-thaw damage of concrete at temperatures below 11 F. Additional recorded, but unpublished, experimental data concerning this phenomenon are available from the same research. To make a valid comparison of different rates of cooling, it is believed that upper and lower limits of the cooling cycles should be the same for all rates involved.

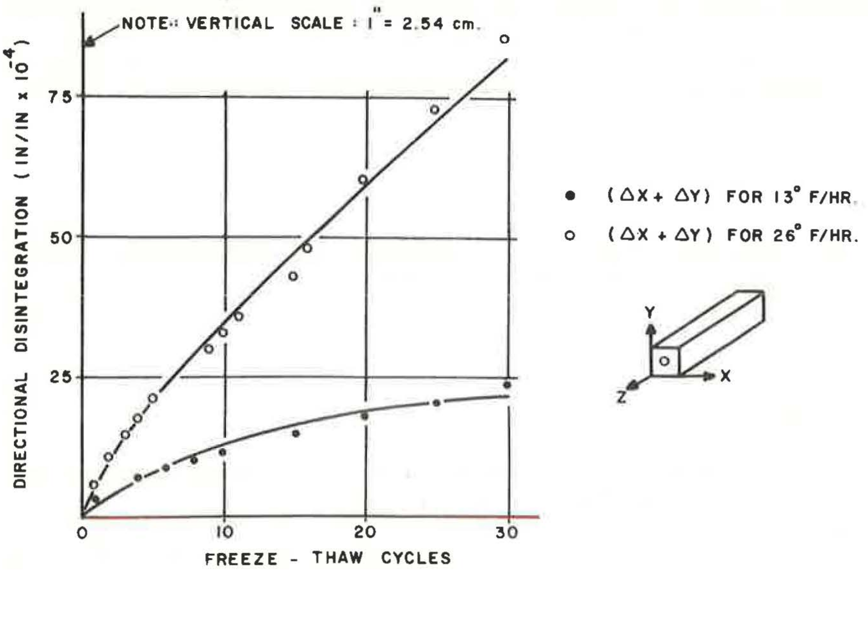
In the authors' research, the concrete specimens were cycled in water from 40 F to 0 F and back to 40 F. In the discusser's research, the concrete specimens were cycled in humid air and then sprayed with water (at 40 F) for 2 hours at the end of the heating cycle. This difference produced an endless supply of water for the concrete in one case and a limited supply of water in the other case. This additional available source of water in the authors' work was forced into the concrete by the freezing and thawing action. In the discusser's work, it is conjectured that the phenomenon involved was mainly the movement of water from one point to another, with a limited source of water. These entirely different experimental techniques probably produced different results and may represent an area of further research.

After the original research paper was submitted to the Transportation Research Board, the results on research regarding the effects of compressive stress fields on the deep-seated (aggregate-generated) type of concrete deterioration (freeze-thaw damage) were obtained. Graphs from the results of that project illustrate the effects of the rate of cooling on these aggregates in concrete when vacuum-saturated and cooled at 2 different rates (14, 15).

Approximately the same materials and concrete mixture were used in this project as described in the main paper. The same automatic freezer was used for both projects. These members (beams) were 3 × 3 × 14 in. A concentric compressive stress of 1,000 psi was applied in the longitudinal (14-in.) direction of all members. The heat-transfer medium was water-saturated kerosene, and the members were cycled in a closed system, i.e., cooled and heated in water-saturated kerosene. The temperature in the concrete was cycled from 40 F to 0 F and back to 40 F. Cooling rates were 13 F/hour and 26 F/hour, and the heating rates were the same for both conditions. Deterioration was measured by permanent strain or change in length.

Because the members were restrained from elongating in the Z-direction (Figure 5), the permanent change in length was measured in the X- and Y-directions and called "directional disintegration". In Figure 5, the symbol Δ was used to indicate a change

Figure 5. Directional disintegration for 2 cooling rates versus freeze-thaw cycles.



in length in any direction. The numerical value (in inches per inch) for each data point was obtained from 3 beams and was the average value over 36 inches.

The results are shown in Figure 5. At the end of 30 cycles, the directional disintegration at the faster cooling rate (26 F/hour) was approximately 350 percent greater than the directional disintegration at the cooling rate of 13 F/hour. Also, the slope of the curves delineates the rate of the disintegration of the concrete.

There is the possibility that the increase in the rate of deterioration of this concrete could be affected by 2 temperature differentials during the cooling cycles. The freezing front (freezing line) moves from the surface inward; consequently, the faster the cooling rate, the greater the temperature differential in the concrete. This could result in detrimental tensile stresses at the surface. If the temperature differential of the 26 F/hour cooling rate were a contributing variable, the microcracks and the macrocracks would have first appeared on the surface. Physical observation and experimental measurements indicated that the microcracks propagated from the inside to the surface, and not the reverse.

These results are in agreement with Powers' hypothesis (6) in regard to the effect of the rate of cooling on critically saturated particles or aggregates. In this case, the hydraulic pressure generated is a function of the rate of cooling.

The paper illustrates the effects of the rate of cooling on the movement (transportation) of water through the concrete and the rate at which the particles become critically saturated. There appears to be no conflict between the results of the 2 experiments.

From the results in the paper, Figure 2, there is an indication that the transverse fundamental frequency method (ASTM C215-60) may not be the most sensitive method of measuring disintegration of concrete due to these particles when they are critically saturated.

It is possible that for this type of concrete damage a slow cooling rate produces a more rapid movement of water into the concrete and that, once the point of critical saturation is reached, then the higher cooling rate creates the greater damage. This seems to be true in laboratory research and could indicate a field physical phenomenon.

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