

Paul Klieger Symposium on Performance of Concrete

David Whiting
Editor

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Documents

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Documents

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PREFACE

The Paul Klieger International Symposium on Performance of Concrete, co-sponsored by Committees 201 on Durability of Concrete and 222 on Corrosion of Metals in Concrete, was held at the ACI Fall Meeting in San Diego, California in November, 1989. At the opening session I had the pleasure and personal honor of dedicating the symposium to Mr. Paul Klieger. Although Mr. Klieger's many accomplishments are very familiar to his numerous friends and acquaintances in ACI, a brief biographical sketch precedes the preface. Mr. Klieger has been very active in the work of Committee 201, and his knowledge of both experimental findings and practical applications has been invaluable to the committee's activities.

A total of 24 papers was presented at the four sessions of the symposium. The speakers addressed topics ranging from fundamental laboratory studies of concrete durability to case histories of concrete rehabilitation. This volume has been arranged in three parts. Part I covers the more fundamental aspects and laboratory investigations. Part II covers field studies where concrete is exposed to natural conditions. Part III covers case histories of the performance and rehabilitation of concrete structures in severe service environments.

The symposium was noted for its international flavor and for the variety of topics addressed. These included, among others, freeze-thaw resistance, deicer scaling, permeability, corrosion of reinforcement and prestressing, sulfate resistance, microbiologically-induced deterioration, and performance in marine environments. The authors are to be congratulated on the clarity of their presentations and their prompt response to the many requests from the symposium chair.

I would like to thank Mr. Steven Gebler for serving as co-chairman of this symposium, and Messers David Stark and Bernie Erlin for serving as session chairmen. I would also like to thank the many members of committees 201 and 222 who helped review the manuscripts which make up this volume.

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Mr. Paul Klieger was born in Milwaukee, Wisconsin in 1916. He completed his studies at the University of Wisconsin and received a Bachelors Degree in Civil Engineering in 1938.

Subsequent to graduation, Mr. Klieger served as a research assistant in the Mechanics Department, as a materials inspector for the state of Wisconsin, and as an assistant engineer with the U.S. Department of Agriculture. In 1941 he joined the Portland Cement Association, Chicago, Illinois and, with the exception of military service from 1942 to 1945, was employed by PCA until retiring in 1986.

While at the PCA, Mr. Klieger served in the capacities of associate research engineer, senior research engineer, manager of field research, manager of applied research, and director of concrete materials research. He was responsible for much of the work of PCA on durability of concrete, with special emphasis on frost resistance, resistance to deicing chemicals, and sulfate resistance.

Mr. Klieger has received many awards, including the ACI Distinguished Service Award in 1970, the ASTM Award of Merit in 1970, the ASTM Frank E. Richart Award in 1977, and the ACI Arthur R. Anderson Award in 1980. Mr. Klieger is a Fellow and Honorary Member of ACI and also a Fellow and Honorary Member of ASTM. He has served on ACI and ASTM committees too numerous to mention here, and is a former Chairman of ACI Committee 201 on Durability of Concrete.

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How to Make Concrete that will be Immune to the Effects of Freezing and Thawing

by B. Mather

Synopsis: Concrete will be immune to the effects of freezing and thawing if (1) it is not in an environment where freezing and thawing take place so as to cause freezable water in the concrete to freeze, (2) when freezing takes place there are no pores in the concrete large enough to hold freezable water (i.e., no capillary cavities), (3) during freezing of freezable water, the pores containing freezable water are never more than 91 percent filled, i.e., not critically saturated, (4) during freezing of freezable water the pores containing freezable water are more than 91 percent full, the paste has an air-void system with an air bubble located not more than 0.2 mm (0.008 in.) from anywhere ($L \leq 0.2$ mm), sound aggregate, and moderate maturity. Sound aggregate is aggregate that does not contain significant amounts of accessible capillary pore space that is likely to be critically saturated when freezing occurs. The way to establish that such is the case, is to subject properly air-entrained, properly mature concrete, made with the aggregate in question, to an appropriate laboratory freezing-and-thawing test such as ASTM C 666 Procedure A. Moderate maturity means that the originally mixing water-filled space has been reduced by cement hydration so that the remaining capillary porosity that can hold freezable water is a small enough fractional volume of the paste so that the expansion of the water on freezing can be accommodated by the air-void system. Such maturity was shown by Klieger in 1956 to have been attained when the compressive strength reaches about 4,000 psi.

Keywords: age; aggregates; air-entrained concretes; air entrainment; capillarity; compressive strength; concretes; freeze-thaw durability; porosity; saturation; soundness; voids; water

Bryant Mather is Chief, Structures Laboratory, Waterways Experiment Station, U.S. Army Corps of Engineers, Vicksburg, Miss., and has been with the Corps since 1941. He is an Honorary Member and a past president of ACI and is a member of several technical committees. He is currently chairman of Committee 116, Terminology and Notation.

INTRODUCTION

Most of the concrete that has ever been made in the world was made and used in environmental circumstances in which the issue of whether or not it would be adversely affected by freezing and thawing was irrelevant since the environmental exposure either did not involve freezing and thawing or it could be assumed with confidence that, when the concrete froze, it would not be critically saturated. It is not necessary to know how to make concrete that will be immune to the effects of freezing and thawing if the concrete that one is interested in making will never be exposed to freezing and thawing when in a critically saturated condition.

Concrete that is used in environmental situations in which it is exposed to freezing and thawing while critically saturated and which is unaffected by such exposure has been made for probably as long as concrete has been made. Such inadvertently or unintentionally frost-resistant concrete has been made using aggregates that were frost resistant, with cement paste that was air entrained, or was concrete that, in spite of becoming critically saturated, had capillary porosity sufficiently low that the 9 percent expansion of the freezable water could be accommodated by the elastic and creep strain capacity of the concrete without rupture.

So far as I can tell from such literature as I have examined, up until about 50 or 60 years ago most people concerned with resistance of concrete to freezing and thawing simply used the rule of trying to figure out what materials and proportions and construction practices had been used previously with success in a comparable exposure and attempted to duplicate or at least simulate those same materials and proportions and practices in the new concrete; without understanding, in any particular detail, why those aggregates, a cement of that composition, and a concrete produced in the way a particular concrete was produced, yielded "durable" concrete.

This symposium honors Paul Klieger. With that in mind, I looked at the index of the first 227 Bulletins of the Research Department of Portland Cement Association Research and Development Laboratories, published in July 1969, and I observed that 18 Bulletins are listed in the author index following the entry "Klieger, Paul." I then looked at the titles and abstracts of

each of these, beginning with Bulletin 24 in 1949, which deals with the effect of entrained air on a number of properties of concrete including freezing-and-thawing resistance, all the way down to Bulletin 218 in 1967 which deals with laboratory studies of blended cements for their effects on properties of concrete including freezing and thawing. So far as I could tell, 15 of the 18 Bulletins listed were concerned wholly or in part with freezing and thawing. As I shall emphasize later, it was Klieger's 1956 Highway Research Bulletin No. 150 paper that became PCA Research Bulletin 82 (Klieger, 1956) that was called "Curing Requirements for Scale Resistance of Concrete" that contributed for the first time, at least that I ever took in, the third critical element to the recipe on how to make concrete that will be immune to the effects of freezing and thawing.

In the development of this topic, I shall state the conclusion first and then discuss some of the backup and derivation.

RECIPE FOR CONCRETE THAT WILL BE IMMUNE TO THE EFFECTS OF FREEZING AND THAWING

There are two ways of stating the recipe, depending upon how much information the audience being addressed is presumed to be qualified to assimilate. If the audience is not presumed to be able to assimilate much information and should not be given the opportunity to exercise much judgment, then the recipe might properly take the form of saying "Follow the guidance and specification requirements stated in appropriate applicable guides and specifications." What this will then come down to is that the recipe will require that the cementitious materials, the aggregates, the mixing water, and the admixtures will be required to be tested to determine compliance with the appropriate options of the applicable ASTM or other governing standards, that the mixtures shall be proportioned according to the applicable portions of the ACI Manual of Concrete Practice (ACI 211.1) and the materials will be handled, batched, conveyed, mixed, placed, compacted, finished, and cured, all also in accordance with applicable documents in the ACI Manual of Concrete Practice. One of the critical features of this recipe will be that someone must decide whether or not it is practical to have job-site testing for air content of fresh concrete. If so, under normal conditions, the cementitious materials used will be nonair-entraining and the air-void system in the cement paste in the concrete will be obtained by the use of an air-entraining admixture, the appropriateness of the dosage of which will be confirmed by job-site testing for air content of fresh concrete. If such tests are not practical and air-entraining cement is available, then air-entraining cement meeting ASTM C 150 for Type IA, Type IIA, Type IIIA, or ASTM C 595 for Type I(SM)-A, Type IS-A, Type I(PM)-A or Type IP-A, Type IS-A(MS) or Type IP-A(MS) will be specified. The

aggregates will be required to meet ASTM C 33 which will cause only frost-resistant aggregate to be used and the mixing water and the admixtures will be in accordance with appropriate specifications to ensure that they contribute what they need to contribute and do not interfere with the achievement of the appropriate levels of relevant properties produced by the interaction of everything else that is going on.

Under the second alternative, where the audience is perceived to be qualified to understand technical considerations and exercise some level of judgment, the recipe for concrete that will be immune to the effects of freezing and thawing may take a somewhat different form. Instead of discussing whether one shall use air-entraining cement or nonair-entraining cement and an air-entraining admixture, the recipe may instead require that the paste portion of the concrete shall contain an air-void system characterized by a bubble-spacing factor (L) not greater than 0.008 in. (0.2 mm) (ACI 201, 212); which is to say simply that regardless of how it is achieved, the cement paste in the hardened concrete shall contain an air bubble not farther away than 0.008 in. (0.2 mm) from anywhere.

Two comments are appropriate at this point. First, this requirement will, under most conditions, be met if air-entraining cement meeting the relevant ASTM cement specifications or an air-entraining admixture meeting ASTM C 260 is used at a dosage sufficient to meet the air content of fresh concrete requirements stated in the documents prepared by ACI Committees 201 and 212 in the ACI Manual. The second comment is that the 0.008 in. (0.2 mm) spacing factor is the maximum for concrete of a particular level of permeability through which the freezing isotherm is moving at a given rate as a function of the thermal properties of the concrete and the temperature difference between the near surface concrete and the surroundings. What this means is that, assuming the hydraulic pressure mechanism for frost damage, if the depth to which the temperature drops and the speed with which it drops is great enough or the permeability of the concrete is low enough, then the bubbles may need to be closer together if they are to serve their intended function. If the thermal gradient is steeper, the freezing isotherm will move into the concrete more rapidly and if the concrete is less permeable the excess water will move through it more slowly. One further consideration is that most processes by which the permeability of concrete can be materially reduced are also processes by which its porosity is concomitantly significantly reduced. Therefore, all other things being equal, if it is harder for the water to get through the concrete there will also usually be substantially less water going through it as a function of whatever mechanism is presumed to be causing water to move or need to move in connection with the phenomenon of freezing. Finally there is the point that the liquid in a capillary pore is a solution of given concentration before any of it freezes. When some of it freezes the solution concentration of the rest increases and it has a lower freezing

point thus it will only freeze when a lower isotherm comes by.

The issue of aggregates may very well be dealt with, not simply by requiring with compliance of ASTM C 33, but rather by requiring that the aggregates under consideration be combined into a concrete mixture using either a standard air-entrained cement paste or the cement paste such as that likely to be used in the construction under consideration. Once the concrete has been proportioned, mixed, formed into specimens, finished, and cured, any of several testing procedures may be employed. Perhaps the most widely used is ASTM C 666 Procedure A, which involves subjecting prismatic specimens to laboratory freezing and thawing while the specimen is immersed in water or ice and determining, at selected intervals, the degree to which such treatment has affected some measurable property of the concrete. The most widely used property is fundamental resonant frequency of vibration from which dynamic Young's modulus of elasticity may be calculated. However, alternatively or additionally, the specimens may be tested for pulse velocity, length change, or change in mass, or they may be sacrificed periodically to measure flexural strength or tensile strength or compressive strength in comparison with companion specimens moist cured to the same age. In addition to ASTM C 666, there is also ASTM C 671, the method for critical dilation of concrete specimens and its companion ASTM C 682, the practice for evaluation of frost resistance of coarse aggregate in air-entrained concrete by critical dilation. This pair of methods, derived from recommendations originated by T. C. Powers (1955) and used very effectively by Bailey Tremper in the California DOT (Tremper and Spellman, 1961), and later advocated strongly by Alan D. Buck (Buck, 1976) at the Waterways Experiment Station, permits the measurement of the time required for a specimen of given initial moisture condition to achieve both critical saturation and, consequently, susceptibility to damage by freezing and thawing if the concrete is intrinsically not frost resistant either by reason of containing nonfrost-resistant aggregate or a paste of inadequate air-void system or a fractional volume of freezable water greater than can be accommodated by the air-void system which is a consequence of inadequate maturity at the time of freezing. Finally, there is the procedure ASTM C 672 for scaling resistance of surfaces exposed to deicing chemicals, by the use of which one can measure this specific manifestation of lack of immunity to freezing and thawing. ASTM C 672 is a procedure to the development and use of which Paul Klieger made probably greater contributions than were made by anyone else (Klieger, 1955; 1956; Klieger and Perenchio, 1963; Verbeck and Klieger, 1956). The mechanism by which a particle of porous unsound aggregate generates internal force sufficient to produce a popout if frozen while critically saturated has been elegantly described by Bache and Isen (1968). The procedure for recognizing such particles has been most thoroughly studied by Larson and Cady (1969).

Finally, after one has dealt with the air-void system in the paste and the quality of the aggregate, there is the third critical requirement regarding concrete that is to be immune to frost action and that is maturity and it is this requirement that we owe primarily to Paul. In his 1956 Highway Research Board paper, which became PCA Bulletin 82 (Klieger, 1956), one of his conclusions was "the development of a certain level of strength has merit, as an index to the amount of curing required for air-entrained concrete prior to permitting the use of deicers." What he found was that for a number of different concretes made with Types I, II, and III cement and, in the case of the Type I and Type II with and without 2 percent calcium chloride, cured at 73, 40, and 25° F, that it took anywhere from 7 days to more than 60 days of moist curing to develop satisfactory resistance to the form of freezing and thawing damage called "deicer scaling." He then reported the compressive strengths of these concretes and they ranged in general from somewhat over 3,000 to somewhat over 4,000 psi. To be on the safe side, I have recommended that one not allow repeated cycles of freezing and thawing of critically saturated concrete to occur prior to the concrete having achieved a strength of 4,000 psi. As I mentioned, Paul said this in 1956. It took awhile for some of us, especially me, to take it in.

In 1962 I presented a paper at the Highway Research Board on effects of duration of moist curing on freezing and thawing of concrete as measured by ASTM C 666 Procedure A (Mather, 1962). In it results were given of quite a lot of tests on quite a number of different concretes made with and without entrained air at water-cement ratios of 0.5 and 0.8 by mass, subjected to freezing and thawing after 14 and 180 days of moist curing at 73° F. It was clear from these data that nonair-entrained concrete, even at 0.5 water-cement ratio, cured for 180 days, which developed strengths of up to 8,100 psi, never got a durability factor greater than 10. On the other hand, air-entrained concrete of 0.8 water-cement ratio cured only 14 days with compressive strengths under 4,000 psi showed durability factors rarely as high as 60. Lower water-cement ratio (0.5) concrete showed durability factors in excess of 60 even at 14 days because the volume of originally water-filled space was initially much less than in the 0.8 W/C concrete and it was sufficiently reduced by cement hydration so that when that space was filled with freezable water its fractional volume was not greater than that which could be accommodated by the air-void system. Since these data are not readily accessible I present them here. The concretes were made using five portland cements, with and without an air-entraining admixture used to give an air content of 6.0 ± 0.5 percent. The aggregate was 3/4-in. nominal maximum size crushed limestone. Mixtures were made at 0.5 and 0.8 W/C by mass. Three rounds were made using a batch on each of three days for each mixture.

Figure 1 (table 1) compares 20 concretes, 10 made at 0.5 water-cement ratio, 10 made at 0.8 water-cement ratio; 10 with entrained air to give an air content of 6.0 ± 0.5 percent and 10

nonair-entrained; made using the five portland cements; two Type I cements (one of high- and one of low-alkali content), and one each of Types II, III, and IV. As might have been expected, the Type III cement concrete showed the least, and the Type IV cement concrete the greatest increase in strength between 28 and 180 days. The nonair-entrained concretes all showed very little resistance to freezing and thawing; but an apparent trend toward a slight increase in durability factor with increasing age and increasing strength is suggested for those concretes with strengths of 7,000 psi or more. This increase only brings the durability factor up to 10.

The Type III cement concrete showed a reduction of durability factor between 14 and 180 days at both water-cement ratios, and the Type I (low-alkali) cement concrete showed a reduction at the 0.8 water-cement ratio.

Figure 2 (see table 2) shows the behavior of concretes of both water-cement ratios, made with Type II cement, as progressive replacement of the cement, by solid volume, with fly ash is made in three stages. For both water-cement ratios at both ages there is a progressive reduction of both strength and durability factor as the proportion of cement replaced by fly ash increases.

Figure 3 (table 2) shows similar data for ground granulated iron blast-furnace slag.

Figure 4 (table 3) shows the behavior of concretes of both water-cement ratios made with Type IV cement, as the cement is replaced by selected percentages by solid volume of each of six materials. All concretes made with Type IV cement show an increase in both strength and durability factor with age.

Figure 5 (table 3) shows similar relations for concretes made with the Type I (low-alkali) cement. At the 0.5 water-cement ratio all show an increase in strength and durability factor with age. At the 0.8 water-cement ratio the concrete made with Type I (low-alkali) cement, as noted in Fig. 1, shows a decrease in durability factor with age; such a decrease was also shown when this cement was replaced either by 12 percent uncalcined diatomite, 30 percent calcined shale, 35 percent pumicite, or 50 percent slag; with 45 percent fly ash replacement, there was essentially no change in durability factor with age, and with 35 percent natural cement the durability factor showed a sharp increase with age.

Figure 6 (table 4) compares data on 26 mixtures, all made with Type II cement, to which no pozzolan or special cement was added; 13 at the 0.5, and 13 at the 0.8 water-cement ratio. One mixture at each water-cement ratio is that shown in Fig. 1, and contains entrained air but no other chemical admixtures; all the other mixtures contain other chemical admixtures, eight each contain calcium lignosulfonate, calcium chloride, and triethanolamine.

Figure 7 shows on one graph all the data on air-entrained concrete given in Fig. 1 through 6. Two heavy lines have been drawn indicating an envelope within which essentially all the data points fall. It is suggested, therefore, that for concrete made using the aggregate under study, the compressive strength developed is related to the durability factor within these limits regardless of the age of the concrete (14 to 180 days for durability factor; 28 to 180 days for strength), the nominal water-cement ratio (0.5 to 0.8), the presence of chemical or mineral admixture; or in essence what Klieger showed in 1956, that if you want frost resistance concrete--for example, concrete with a durability factor over 50--you need air-entrained concrete which has no higher a fractional volume of freezable water than that of concrete having matured so as to develop a compressive strength of about 4,000 psi.

One can do some interesting arithmetic if one were to assume a value of the fractional volume of freezable water, consider the 9 percent expansion of this volume, when the water is converted to ice, and the volume of entrained air required to take up that much increase in volume of the freezable water. If one were to assume that there is slightly more than 10 percent freezable water in a given concrete, it would follow that there is 1 percent or so of increase in volume and ideally only 1 percent by volume of entrained air would accommodate this if the air were used with maximum efficiency and were nearly completely compressed. With a 2 percent air content, the air content of each air void would only need to be compressed to one-half of its original volume and the volume increase would be accommodated. However, in fact, one needs 9 percent fractional volume of air in the mortar fraction of concrete in order to achieve successful protection against frost action. This was first emphasized by Klieger in his paper that became PCA Bulletin 77 (Klieger, 1956A). The conventional assumption is that this is because a lot of the air bubbles are a lot bigger than they need to be. If they were all of the optimum size and optimum distribution, the total air content would be quite a lot less and the penalty in strength of 5 percent per 1 percent of air could be proportionally reduced.

WHY THE RECIPE WORKS

In the preceding portions of this paper, I have discussed different forms of the recipe for making concrete that will be immune to frost action as a function of the sophistication of the audience to which the recipe is being presented. There is, however, a third audience that not only wants the recipe presented in a form that permits the exercise of judgment through the possession of understanding, but also wants technical and scientific explanations of how and why all these things work. The principal contributors to the understanding of how air entrainment works in doing good things for the resistance of concrete to

freezing and thawing have been T. C. Powers (1949, 1954), Richard Helmuth (Powers and Helmuth, 1953), and Bob Philleo (1986). Contributions by each of them are cited in the list of references. The principal contributors to the understanding of the role played by aggregate properties include also T. C. Powers (1955) but especially George Verbeck (Verbeck and Landgren, 1960). Indeed, the subject of resistance of natural aggregate materials, sand, gravel, and crushed stone, to freezing and thawing has a very long history going back at least to 1818 when Brard in France (DeThury, 1829) developed the prototype of what is now ASTM C 88, the standard test for soundness of aggregate by the use of sodium sulfate or magnesium sulfate. This widely misunderstood and generally reviled procedure has been used to assess the durability of building stone and was spoken of quite favorably by Schaffer (1932). It is not always realized that it was always the intent of this test to simulate the expansion of water being converted to ice on freezing by the use of the mechanism of precipitating hydrated sulfate solids in pores in the rock, dehydrating or partially dehydrating these sulfates by oven drying and causing the internal expansion that simulates ice formation by the rehydration of the dehydrated or partially dehydrated sulfates when the specimen is re-immersed in the sulfate solution. ACI Committee 201 has, in the last few years, given attention to the relationship of this mechanism and the deterioration of concrete associated with the movement of sulfate solutions into concrete by capillary action, the subsequent precipitation of salt by dehydration and damage due to rehydration. In this case, the test that was to simulate freezing and thawing also reproduces a completely unrelated phenomenon which may cause destruction of concrete by internal expansion in regions where freezing cannot occur.

CONCLUSION

Concrete will be immune to the effects of freezing and thawing even if critically saturated with water if it is made using sound aggregate, has a proper air-void system, and has matured so as to have developed a compressive strength of about 4,000 psi.

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C 150 Specification for Portland Cement

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- C 457 Practice for Microscopical Determination of Air-Void Content and Parameters of the Air-Void System in Hardened Concrete
- C 595 Specification for Blended Hydraulic Cements
- C 666 Test Method for Resistance of Concrete to Rapid Freezing and Thawing
- C 671 Test Method for Critical Dilation of Concrete Specimens Subjected to Freezing
- C 672 Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals
- C 682 Practice for Evaluation of Frost Resistance of Coarse Aggregates in Air-Entrained Concrete by Critical Dilation Procedures

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Table 1
Strength, Age, Durability Data for Five Cements

Cement	w/c = 0.5				w/c = 0.8			
	Compressive		Compressive		Compressive		Compressive	
	Strength	DFE	Strength	DFE	Strength	DFE	Strength	DFE
	psi	14	psi	180	psi	14	psi	180
	28 days	days	180 days	days	28 days	days	180 days	days
6 ± 0.5% Air								
II	4440	90	6180	92	1990	49	2770	58
I-H	3350	79	4710	88	1630	17	2070	40
I-L	4840	91	5740	92	2520	59	2830	50
III	6110	93	6610	91	2990	67	3220	64
IV	3760	67	6320	89	1420	29	2880	63
Nonair-Entrained								
II	5620	3	7710	5	2670	2	3860	2
I-H	4340	2	5850	3	1900	2	2400	2
I-L	6610	5	7430	9	2800	2	3620	1
III	6750	5	8120	10	3550	3	3960	3
IV	4070	2	6940	7	1840	2	3790	1

Table 2
Strength, Age, Durability Data for Type II Cement
Two Mineral Admixtures

Admixture	% Cement Replaced by Vol	w/c = 0.5				w/c = 0.8			
		Compr		Compr		Compr		Compr	
		Str	DFE	Str	DFE	Str	DFE	Str	DFE
		psi	14	psi	180	psi	14	psi	180
		28 days	days	days	days	28 days	days	days	days
Fly ash I	30	3510	66	5540	85	1180	23	2200	22
Fly ash I	45	2810	51	5000	72	1130	12	2150	14
Fly ash I	60	2060	35	3590	49	770	5	1650	3
Slag I	30	3890	83	5230	91	1840	29	2500	36
Slag I	50	3030	72	4920	83	1790	18	2500	19
Slag I	70	2370	53	3610	39	1320	3	1860	4

Table 3
Strength, Age, Durability Data for Two Cements
Six Mineral Admixtures

		w/c = 0.5				w/c = 0.8			
Admixture	% Cement Replaced by Vol	Compr		Compr		Compr		Compr	
		Str	DFE	Str	DFE	Str	DFE	Str	DFE
		psi	14	psi	180	psi	14	psi	180
		28 days	days	days	days	28 days	days	days	days
Type IV Cement									
Slag I	50	2870	62	4850	76	1500	6	2540	13
Nat I	35	3830	61	6580	89	1490	18	2760	43
Fly ash I	45	2140	36	4640	67	870	2	2310	6
Pum F	35	2260	61	5280	83	880	13	2430	21
Cal sh M	30	2890	56	5430	91	1250	21	2810	43
Unc diat	12	3840	66	6260	94	1420	29	2780	64
Type I (Low-Alkali) Cement									
Slag I	50	3810	66	5290	85	1690	15	2370	12
Nat I	35	4750	85	6180	91	1840	31	2540	55
Fly ash I	45	3080	58	4990	77	1190	9	2170	10
Pum F	35	3390	72	5380	84	1300	34	2160	19
Cal sh M	30	4190	76	5240	90	1960	55	2170	40
Unc diat	12	4910	89	6060	94	2340	63	2710	60

Table 4
Strength, Age, Durability Data for Type II Cement
Three Chemical Admixtures

Admixture	w/c = 0.5				w/c = 0.8			
	Compr		Compr		Compr		Compr	
	Str psi 28 days	DFE 14 days	Str psi 180 days	DFE 180 days	Str psi 28 days	DFE 14 days	Str psi 180 days	DFE 180 days
None	4440	90	6180	92	1990	49	2770	58
CLS 73*	4860	90	6700	92	1930	36	2690	50
CLS 73**	5310	92	6140	89	2230	59	2910	52
CLS 100*	4940	91	6650	91	2010	51	2720	72
CLS 100**	5290	88	5910	79	2700	54	3010	33
CaCl ₂ 73*	4810	89	6420	91	2080	48	2900	60
CaCl ₂ 73**	5300	91	6110	88	2350	55	3120	44
CaCl ₂ 40*	5340	91	7180	90	2200	43	3060	73
CaCl ₂ 40**	4790	97	6730	89	1900	59	3290	75
Trieth 73*	5110	89	6190	91	2530	48	3190	61
Trieth 73**	4760	90	5610	83	2640	50	3220	45
Trieth 40*	5300	90	6580	89	2400	54	3250	72
Trieth 40**	4670	98	6510	89	2000	68	3590	81

* Casting temperature.

** Casting-and-curing temperature.

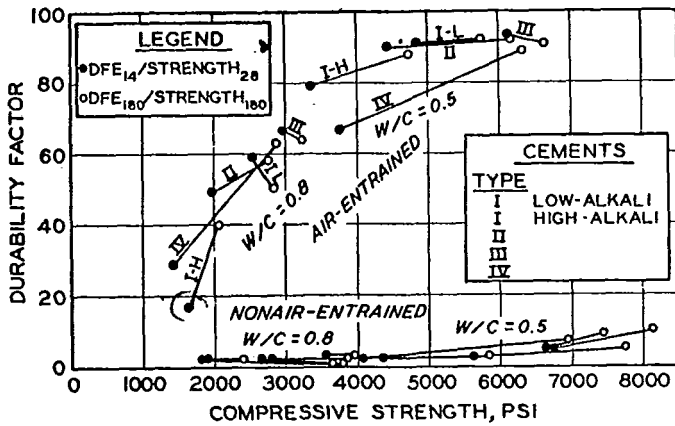


Fig. 1--Comparison of durability factors and compressive strengths of nonair-entrained and air-entrained concretes

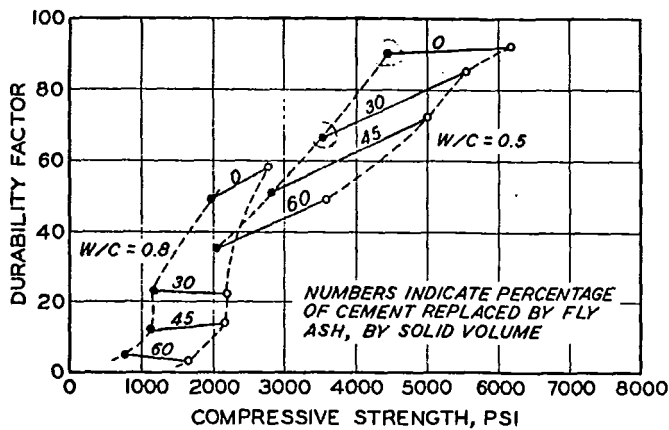


Fig. 2--Effect on durability factors and compressive strengths of replacing type II cement with fly ash I

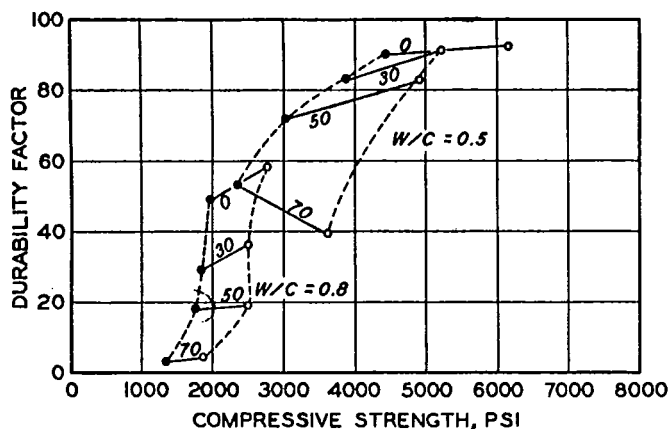


Fig. 3--Effect on durability factors and compressive strengths of replacing type II cement with slag I

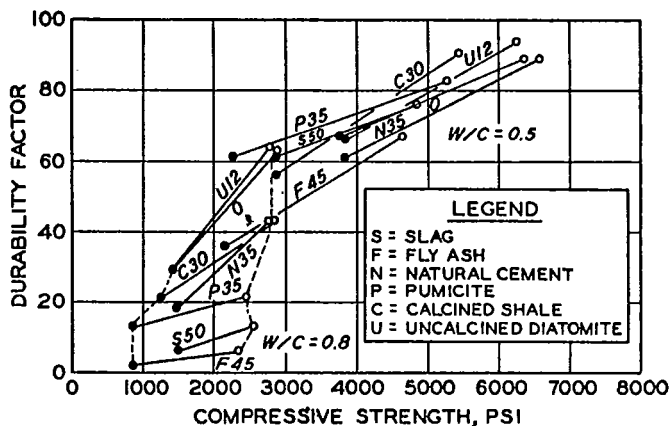


Fig. 4--Effect on durability factors and compressive strengths of replacing type IV cement with six materials

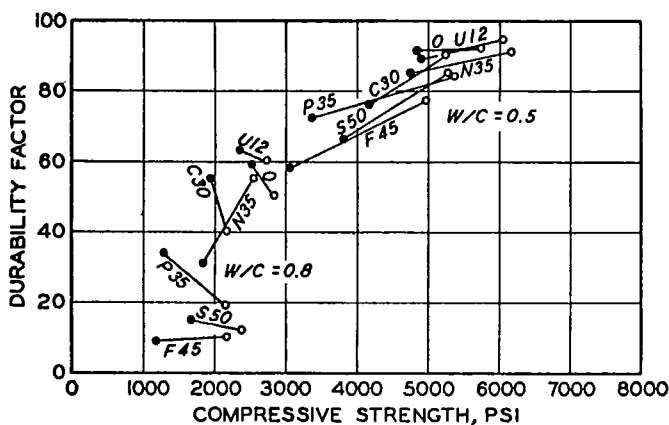


Fig. 5--Effect on durability factors and compressive strengths of replacing type I (low-alkali) cement with six materials

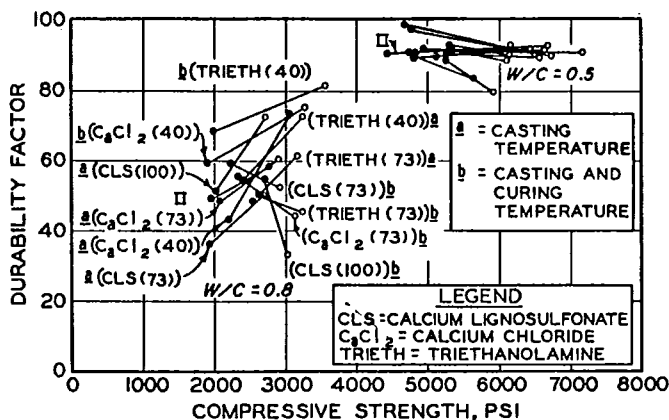


Fig. 6--Durability factors and compressive strengths of concretes made with type II cement with and without chemical admixtures

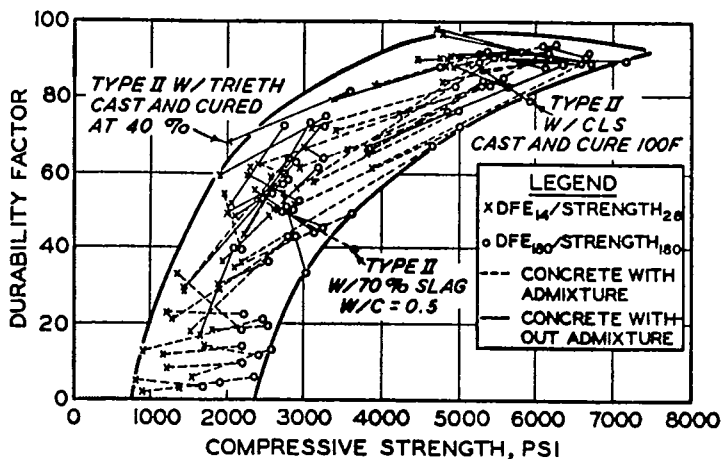


Fig. 7--Summary of data for air-entrained concrete from Figs. 1-6

SP 122-2

Durability of High Strength Concrete

by P. K. Mehta

Synopsis: The commercial utilization of high-strength concrete with 60-120 MPa compressive strength is a recent phenomenon, therefore long-term field experience in regard to durability to corrosive environments is not available. In this paper a critical review of the factors necessary to obtain high strength and high durability is presented. Typically, the concrete mixtures contain high cement content, low water content and several admixtures, such as a superplasticizer, a pozzolan, and at times an air-entraining agent. When properly placed, consolidated, and cured, such mixtures should have low permeability and high durability to corrosive environments. However, there is some concern that microcracking in the aggregate-cement paste transition zone, possibly due to a variety of causes, may impair the impermeability and durability. The results of a recent investigation are discussed, which show that the aggregate type can play an important role in controlling the strength of the transition zone and, therefore, the degree of potential microcracking of concrete in service.

Keywords: aggregates; concrete durability; high-strength concretes; hysteresis; microcracking; modulus of elasticity; permeability

P. Kumar Mehta, FACI, is Professor in the Civil Engineering Department of the University of California at Berkeley. He is the author of numerous papers including a text on concrete, and was awarded ACI's Wason Medal for Materials Research in 1988.

INTRODUCTION

The uniaxial compressive strength of ordinary structural concrete ranges typically from 25 to 35 MPa. During the last 10 years commercial utilization of 60 to 120 MPa concrete mixtures has been reported from various metropolitan areas of North America; concrete with more than 40 MPa compressive strength is, generally, called **high-strength**.

To produce high-strength concrete, special attention must be paid to mix proportioning, selection of materials, and concreting practice. Since there is an inverse relationship between porosity and strength, the microstructure of the composite material or concrete has to be engineered so as to obtain as low a porosity as possible in all the component phases that make up the concrete microstructure. It should be noted that from standpoint of microstructure-property relationships, the three components of the microstructure of concrete are cement paste (or mortar), aggregate, and the transition zone surrounding coarse aggregate particles (1).

Dramatic reductions in porosity of the bulk mortar and the transition zone have become possible by the use of superplasticizing and mineral admixtures in concrete. Most aggregates from natural sources are dense and strong (100 to 300 MPa crushing strength) and, in ordinary concrete mixtures, they seldom become strength limiting. However, when designing very high-strength concrete mixtures it should be obvious that the aggregate strength can no longer be ignored. Using high quality aggregate, high cement content, and a very low water/cementitious ratio (0.2 to 0.3), concrete mixtures can be produced which are essentially free from capillary pores as well as microcracks in the bulk mortar and the transition zone and, therefore, give very high compressive strengths.

The above microstructural characteristics, that are responsible for high strength in concrete, are also essential in obtaining a low permeability which is the most needed property when a concrete structure is to be exposed to an aggressive environment.* In laboratory tests, high-strength concrete mixtures have shown excellent durability to various physical and chemical forms of attack. For instance, Wolsiefer (2) reported that a 120-MPa concrete showed no permeability at all to water under 4 MPa pressure, very low chloride absorption, only slight salt scaling after 500 cycles in the ASTM C 672 test, and negligible abrasion in the ASTM C 779 test. Laboratory tests for evaluation of concrete durability generally fail to simulate the behavior of concrete in field practice,

*Low permeability alone is a sufficient criterion for durability of concrete to most aggressive environments, however it is assumed that proper air entrainment will also be needed when a concrete structure is to be protected against cycles of freezing and thawing.

therefore most of our knowledge regarding the factors that are important in durability of concrete is derived from long-time field tests and case histories of deteriorated concrete in real structures. Since high-strength concrete is a relatively recent material of construction, results are not available from any long-time durability tests and from field behavior structures. However, there is enough available information which, when taken into consideration, should help us build durable structures with high-strength concrete. In this paper, an attempt is made to present a brief review of factors that are considered important from standpoint of durability. Special attention is invited to the sections dealing with aggregate selection and curing technology, which contain the results of recently concluded research studies at the University of California at Berkeley.

INFLUENCE OF MATERIALS AND MIX PROPORTIONS ON PERMEABILITY

It is generally agreed that concrete mix proportions have probably the greatest influence on concrete permeability. This is because, at a given degree of cement hydration, there is a direct relationship between the water/cement ratio and the volume fraction of capillary pores in hardened cement paste on one hand, and between the capillary porosity and the coefficient of permeability on the other hand. For example, as is illustrated by Fig. 1, a cement paste with 0.3 water/cement ratio and 50% degree of hydration will contain approximately 20 percent capillary pores and will have a very low coefficient of permeability (1). It seems that in a hydrated cement paste there should be at least 30% capillary porosity before sufficient interconnections can be established between the pores to cause some fluid flow. Note that the shaded area in Fig. 1 shows capillary porosity and permeability ranges for portland cement pastes in ordinary concrete.

In field practice, high-strength concrete mixtures of adequate workability, with water/cement ratios of the order of 0.3 or less, cannot be made without the use of superplasticizers. It is well known that due to the presence of residual surface charges, portland cement particles when mixed with a polar liquid, such as water, tend to flocculate trapping a considerable amount of the liquid in the flocs and, thus reducing the workability of concrete. With ordinary concrete considerably larger amounts of water than needed for cement hydration are used to overcome this difficulty. In the case of high-strength concrete mixtures the use of surfactants to break up the cement-water flocs is the preferred alternative for obvious reasons. However, normal water-reducing admixtures (such as lignosulfonates), when used in large dosages than are usually necessary to obtain workable concrete mixtures with less than 0.3 water/cement ratio, tend to produce certain undesirable side-effects. Formaldehyde sulfonates of naphthalene or melamine, known as superplasticizers, are found to be highly effective surfactants, and have become the workhorse of the high-strength concrete industry.

Besides the beneficial effect of a superplasticizer on the density and impermeability of hydrated cement paste, as discussed above, there is yet another component of the concrete microstructure (namely the transition zone between mortar and coarse aggregate particles) which is greatly strengthened by the use of low water content that is typical of high-strength concrete mixtures. With ordinary concrete mixtures, as a result of high water content, the internal sedimentation and bleeding effects in the vicinity of coarse aggregate particles cause the transition zone to be porous and weak. A weak transition zone is vulnerable to microcracking easily when the concrete is subjected to normal loads and environmental stresses (i.e. thermal and humidity cycles). In fact, it is the presence of interconnected network of microcracks around the coarse aggregates in concrete which is assumed to be responsible for the phenomenon that, at a

given water/cement ratio, the permeability of the concretes is generally higher than the permeability of the corresponding cement paste by two orders of magnitude.

A superplasticized concrete mixture, with a low water content and no bleeding tendency, has therefore the potential of becoming virtually impermeable. However, there is one problem. The superplasticized concrete usually tends to show rapid slump loss; and, in order to regain the desired workability, it becomes necessary to retemper the mixture with either more water or more superplasticizer or both. It is not uncommon to find that this solution to the problem of slump loss creates another problem, namely the tendency for segregation and bleeding, which would have an adverse effect on strength and durability. Since the incorporation of finely divided particles of cementitious or pozzolanic materials (mineral admixtures) into a superplasticized concrete mixture provides an excellent solution to the bleeding problem, it is almost imperative to use a mineral admixture in combination with a superplasticizer whenever the durability of concrete is of primary concern. It should be noted that, when their quality and quantity in a concrete mixture are adequate, the mineral admixtures not only have the ability to enhance the workability and the impermeability but also the resistance of concrete to cracking by thermal stresses, alkali-silica reaction, and sulfate attack.

Finally, the maximum size of aggregate is also known to have a great influence on the permeability of concrete. The two independently obtained sets of data shown in Fig. 2 clearly illustrated that the coefficient of permeability would increase not only with the increasing water/cement ratio but also with the increasing size of aggregate in concrete (1). The explanation for this behavior lies again in the higher degree of microcracking in the transition zone with increasing aggregate size. The aggregate size and grading have a great influence on the bleeding characteristics of a fresh concrete mixture. This, in turn, affects the strength and the microcracking tendency of the transition zone. Since the microcracks originating at the transition zone tend to propagate both in width and length due to various kinds of stress effects on concrete structures in service, they become the primary source of permeability. Therefore, to reduce the potential of excessive microcracking in high-strength concrete mixtures, it is desirable to limit the maximum size of aggregate to 12-15 mm. Incidentally, small aggregate size is also beneficial for high strength and it is a common practice to use small aggregate in high strength concrete mixtures.

OTHER FACTORS INFLUENCING THE DURABILITY OF HIGH-STRENGTH CONCRETE

Based on the principles discussed above, a typical high-strength concrete mixture should, intrinsically, have a very low coefficient of permeability and high durability to corrosive environments. In fact, this has been confirmed by numerous laboratory investigations. However, in field practice sometimes the expected performance result may not be obtained. This is because there are factors other than the characteristics of concrete-making materials and mix proportions which govern the permeability of concrete in service. Most of these factors have to do with field operations involving batching, mixing, transportation, placement, consolidation, and curing of concrete. A detailed discussion on the concreting operations is beyond the scope of this paper. Briefly, when a heterogeneous concrete mixture results due to incomplete mixing, improper placement, poor consolidation, or inadequate curing, usually it is the areas of microstructural inhomogeneity which turn out to be the potential sites for crack initiation.

RECENT STUDIES AT THE UNIVERSITY OF CALIFORNIA, BERKELEY

Effects of the aggregate type and the length of curing period on the characteristics of concrete were recently investigated at the University of California at Berkeley. A summary of the findings, which should be relevant to long-time concrete durability, is given below.

Effect of Curing Period

Asselanis et. al (3) investigated the influence of curing period on the stress-strain behavior and weight changes of concrete specimens made from a high-strength mixture. The concrete mixture contained 472 kg/m³ ASTM Type II portland cement, 47 kg/m³ of condensed silica fume, 0.31 water/cement ratio, a naphthalene sulfonate type superplasticizer (1% solids by wet. of cement), and a 10-mm size crushed diabase as the coarse aggregate. From the compressive strength, elastic modulus, and weight changes data it was concluded that, for high-strength concrete mixtures of this type, a 7-d moist-curing period was sufficient to make the concrete almost impervious. Additional moist-curing beyond 7 days was not necessary to substantially enhance the engineering properties, which showed considerable improvement on air-curing. This is because the concrete, having become impervious in 7 days, retained enough moisture needed for further cement hydration. Longer curing periods are usually desirable with concrete mixtures containing fly ash or slag instead of silica fume.

Effect of Aggregate Type

It was pointed out that with ordinary concrete (< 40 MPa) the strength of the coarse aggregate is seldom an issue. This is because with water/cement ratios in the 0.4 to 0.7 range, it is the hardened cement paste and especially the transition zone which normally are much weaker components in the three-phase system. This is no longer the case with high-strength concrete mixtures. Due to the low water content (< 0.3 water/cement ratio) and lack of bleeding, the transition zone or the aggregate-cement paste bond in high-strength concrete mixtures can be so strengthened that only a few aggregate particles show debonding when a concrete specimen is crushed under uniaxial compression. Unlike ordinary concrete, in high-strength concrete the applied stress may be fully transferable from the cement paste matrix to the aggregate. Under these conditions the strength of the aggregate would begin to play a role in determining the mechanical characteristics of concrete.

To test the above hypothesis, Aïtcin and Mehta (4) tested four locally available high-quality coarse aggregates in a high-strength concrete mixture of the type used by Asselanis et. al (3). One of the aggregates was a natural siliceous gravel, and the other three were crushed rocks of different types, viz. a granite, a diabase, and a limestone. All the aggregate particles were clean, hard, strong, and mineralogically uniform except the granite, which contained inclusions of laumontite in a matrix of quartz and feldspar.

The 28-d and 56-d compressive strengths and elastic moduli of concrete specimens containing either the granite or the siliceous gravel were found to be significantly lower than those containing either the diabase or the limestone. To explain the reasons underlying the differences in behavior, the stress-strain curves and the loading-unloading hysteresis loops are shown in Fig. 3. From the Figure it is clear that the diabase and limestone concretes not only gave significantly higher stress/strain ratio, but also much narrower hysteresis loops. The width of the hysteresis loop is probably related either to the intrinsic elastic property of the aggregate (such as the presence of voids, microcracks, or

inclusions of weak minerals in the microstructure of the aggregate), or to the strength of the transition zone. Since the water/cement ratio and bleeding characteristics of all concrete mixtures were similar, the differences in the strength of transition zone can be attributed to chemical bonding. The diabase and limestone aggregates, being fine-grained, provided more reactive surface with cement paste than the granite and siliceous gravel, which were coarse-grained. A chemical reaction between calcite and calcium hydroxide (cement paste) has also been stipulated in the published literature (5).

Also, it may be noted that with the limestone- or diabase-aggregate concrete the hysteresis loop closed almost completely, whereas this did not happen with the granite- or gravel-aggregate concrete. Again, the presence of a non-recoverable (plastic) strain in the latter is assumed to be due either to defects within the aggregate microstructure or due to a weaker transition zone which is prone to microcracking even at low levels of applied stress (within the elastic range). The latter probability was suspected here since many aggregate particles were found debonded in the granite and gravel concretes. This finding is considered significant from standpoint of durability of high-strength concrete mixtures to be exposed to corrosive environments. For instance, a particular aggregate type may be strong and hard and thus suitable to obtain the desired strength in a high-strength concrete mixture. However if the transition zone in the concrete is prone to microcracking at stress levels encountered in service, then the permeability of concrete is likely to increase and durability is bound to suffer.

CONCLUSION

The permeability of concrete is the most important parameter controlling durability to corrosive environments. With ordinary concrete, the permeability can be reduced by selecting mixtures which involve the use of high cement content, low water/cement ratio, water-reducing agents and mineral admixtures. Since most of the factors cited above are standard requirements with high-strength concrete mixtures, a high-strength concrete will have a low permeability. However, should the concrete develop extensive microcracking in service due to any reasons, the permeability is likely to rise and durability would be impaired. A recent study with high-strength concrete mixtures has shown that the aggregate type exercised a significant influence on strength of the transition zone and its microcracking behavior under stress. Thus from standpoint of durability of high-strength and low-permeability concrete, the selection of aggregate should be made after laboratory studies of the type described in this paper.

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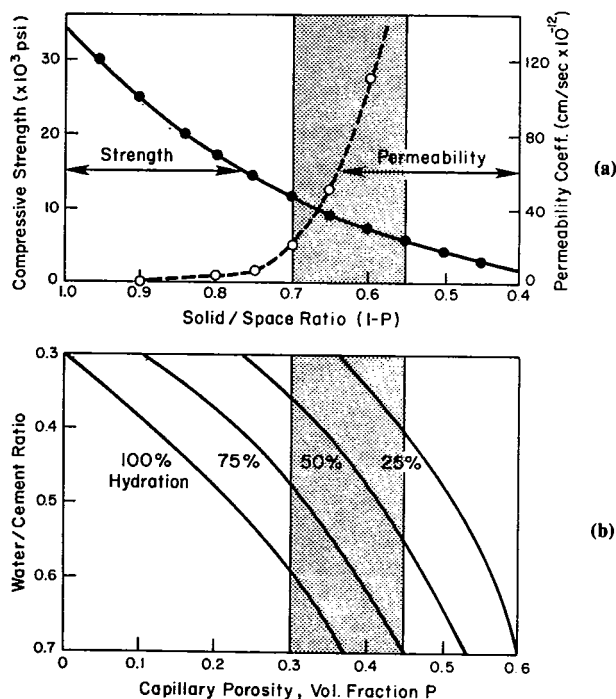


Fig. 1 Influence of water/cement ratio and degree of hydration on strength and permeability.

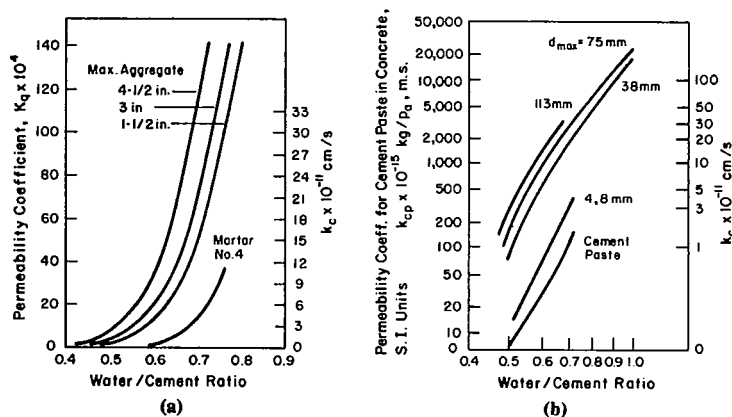


Fig. 2 Influence of water/cement ratio and maximum aggregate size on concrete permeability: (a) K_a is a relative measure of the flow of water through concrete in cubic feet per year per square foot of area for a unit hydraulic gradient. [(a), From Concrete Manual, 8th Edition, U.S. Bureau of Reclamation, 1975, p. 37, (b), adapted from Beton-Bogen, Aalborg Cement Co., Aalborg, Denmark, 1979.]

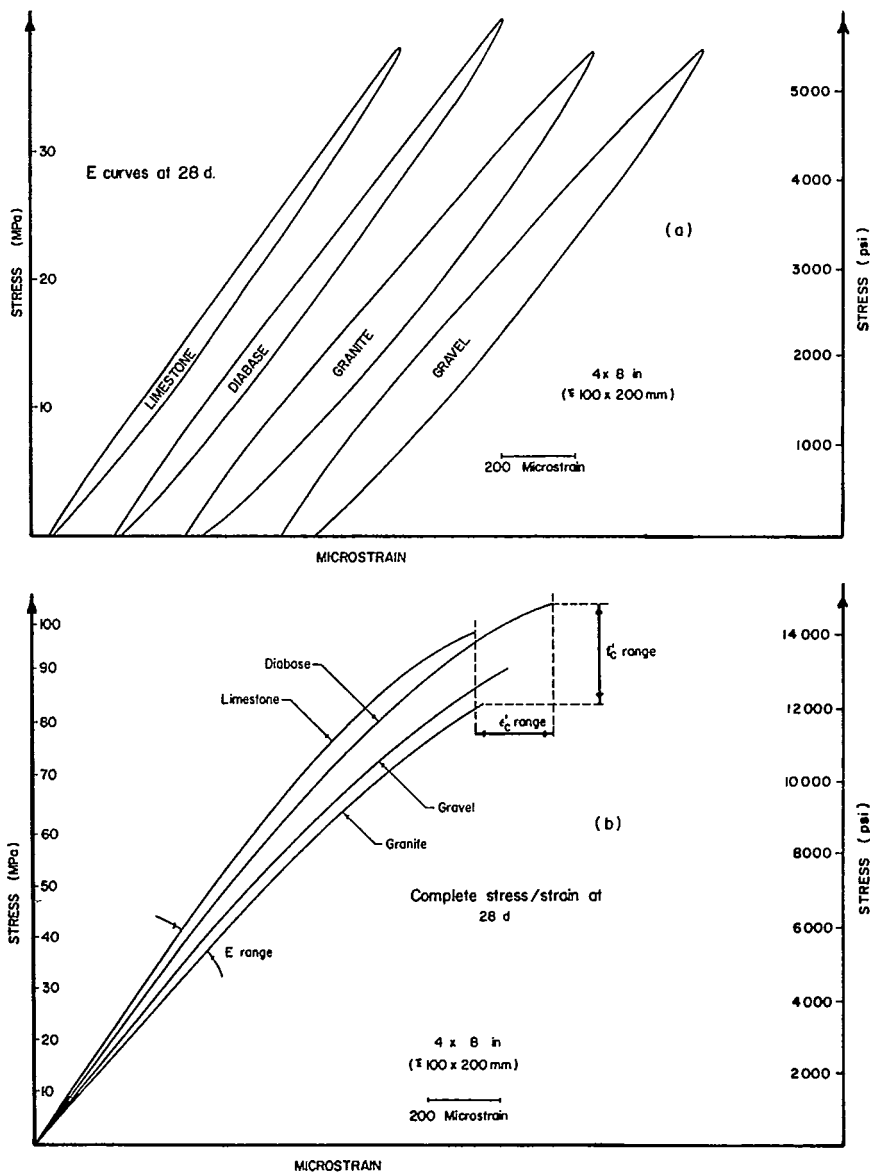


Fig. 3 Influence of aggregate type on the behavior of concrete specimens under uniaxial compression: (a) loading-unloading hysteresis loops in the elastic range, (b) stress-strain curves to failure.

SP 122-3

Deicer Salt Scaling Resistance of High Performance Concrete

by R. Gagne, M. Pigeon, and P.-C. Aitcin

Synopsis: Twenty seven high performance concrete mixes (with 28 day strengths in the 80-100 MPa range) were prepared to evaluate the deicer salt scaling resistance of such concretes after various periods of curing. Three water-cement ratios (0,30, 0,26 and 0,23) were used and for each water-cement ratio a minimum of three mixes were made with different air void systems: one with a spacing factor of approximately 200 μm , one with a slightly higher value and one without any air entrainment. Canadian Type 30 cement with an addition of 6% silica fume was used for all mixes. The coarse aggregate was a 14 mm maximum size, crushed, very dense, dolomitic limestone. The curing period varied between 1 and 28 days. A total of 54 specimens (2 for each test condition) were submitted to 150 daily cycles in accordance with ASTM Standard C 672, using sodium chloride as a deicer. Weight loss was measured to evaluate the deterioration of the concrete surfaces. The scaling resistance was found to be extremely good in all cases, irrespective of the length of curing, water-cement ratio or spacing factor value, weight losses after 150 cycles being always lower than 0,50 kg/m^2 . No correlation was found between the scaling resistance and the spacing factor or the length of curing. Loss of mass was generally concentrated around a few aggregate particles. These results indicate clearly that it is possible to prepare high performance concretes with very good deicer salt scaling resistance without using any air entrainment.

Keywords: air-entrained concretes; compressive strength; concrete durability; curing; deicers; freeze-thaw durability; high-strength concretes; silica fume; water-cement ratio

ACI member **Richard Gagné** is a PhD student at Laval University, Québec City, Canada. He received his MS from that university in 1986. His research interests are in the areas of durability of concrete and high performance concrete.

ACI member **Michel Pigeon** is Professor of Civil Engineering at Laval University, Québec City, Canada. His research interests are in the areas of durability of concrete and concrete repairs. He received his PhD from the Université Pierre et Marie Curie in Paris in 1984.

ACI fellow **Pierre-Claude Aïtcin** is Professor of Civil Engineering at Sherbrooke University, Sherbrooke, Canada. His research interests are the use of industrial by-products in concrete, very high strength concrete and concrete applications in the Arctic. He received his PhD from the Université de Toulouse in 1965.

INTRODUCTION

During the past 10 years, concrete technology research has shown that it is possible to produce, for commercial uses, concretes with very high compressive strengths (>80 MPa) (1,2). Engineers and contractors now need to know if it is necessary to use air entrainment in these concretes as a protection against freeze-thaw cycles, particularly in view of the important reduction in compressive strength associated with the use of air entrainment and the increase in production costs due to the use of lower water-cement ratios to compensate this reduction. In high rise buildings, where high strength concrete is used more and more, this problem does not occur because of the absence of concrete saturation and freezing and thawing cycles. It can not, however, be avoided in concrete structures exposed to water, freeze-thaw cycles and deicing salts, such as bridge decks or highway structures.

The question concerning the need to use entrained air in high strength concrete is not resolved at this time. Some tests have shown that air is required (3,4) but other results have indicated that high strength concrete is not really affected by freeze-thaw cycles (5,6,7). Moreover, very few researchers have studied the deicer salt scaling resistance of high strength concrete. Part of the problem comes from the fact that the physical and mechanical properties of high strength concrete can be very different from one study to another, because there is still no precise rules to establish what concretes can be called "high strength concretes".

A research program on the durability of high strength concrete was initiated at Laval University a few years ago by studying the deicer salt scaling resistance (ASTM C 672) of low water-cement ratio concretes made with Type I Portland cement

(7). It was found that the scaling resistance of such concretes can be very good. However more information is needed concerning the influence of the water-cement ratio, of the curing time, of the type of cement and of the use of silica fume. This paper reports the results of tests carried out to assess the effect of some of these parameters.

TEST PROGRAM

It was decided, in this research program, to study the durability of concretes made with three water-cementitious material ratios: 0.30, 0.26 and 0.23. For all mixes ASTM Type III (CSA Type 30) cement was used and 6% of the cement was replaced by silica fume. With this type of cement and this amount of silica fume, it is possible to produce concretes with very high compressive strengths (2). For each water-cementitious material ratio, the dosage of air entraining agent was varied to produce three categories of spacing factor values: low (100-300 μm), moderate (300-500 μm) and high (500-900 μm).

All concrete specimens were cured by sealing in a number of layers of plastic film. This type of curing is closer to normal field curing than water curing. For each mix, a minimum of two curing periods were selected from the following: 1, 3, 7 and 28 days.

Table 1 gives a brief description of the mixes tested. The first two digits of the mix name indicates the water-cementitious material ratio (0.30, 0.26, 0.23), the third one represents the curing period (1, 3, 7 or 28 days) and the letter indicates the spacing factor category (H-High; M-Moderate; L-Low). A total of 54 concrete specimens (2 per mix and curing period, i.e. 27 different combinations of type of mix and curing period) were submitted to the ASTM C 672 deicer salt scaling test.

MATERIALS, MIX CHARACTERISTICS AND EXPERIMENTAL PROCEDURES

All concretes were made with a Canadian Type 30 Portland cement (ASTM Type III). This cement has a very high Blaine fineness (5450 cm^2/g) and a high C_3S content (63%). The silica fume that was used contains more than 90% silica and was obtained from a plant in Bécancour in the province of Québec.

For all mixes, a granitic sand containing particles of feldspar, quartz and hornblende with a fineness modulus of 2.3 was used. Despite the fineness of this sand (Table 2), all the mixes had acceptable workabilities and normal cement contents (for that type of concrete). The coarse aggregate was a dolomitic limestone containing more than 98% of hard dolomite. These two aggregates were used dry in every mix.

The proportions and some of the fresh concrete properties of the mixes made are summarized in Table 3. For all

mixes, the amount of silica fume was kept constant at 6% of the total mass of cementitious material. For a given water-cementitious material ratio and spacing factor category, some mixes have exactly the same proportions because they were in fact obtained from the same batch (in these cases, the curing period is the only parameter that differentiates one mix from the others).

All concrete mixes were prepared using a naphthalene formaldehyde condensate type superplasticizer and a synthetic detergent type air entraining agent. The liquid to solid ratio in this superplasticizer is 0,59 by weight and the amount of water was taken into account in the computation of the final water-cementitious material ratio of the mixes. Relatively high dosages (27 to 50 ml/kg of cementitious material) were required to maintain a minimum slump of 30 mm because of the low values of the water-cementitious material ratios and also because of the high fineness of the ASTM Type III cement used (5450 cm²/g). Because of the very low water content of the various mixes, the effectiveness of the air entraining agent was highly reduced and about 10 times the manufacturer's recommended dosage for normal concrete was used to produce spacing factors in the 100 to 300 μ m range. A slightly lower dosage was used to obtain spacing factors in the 300 to 500 μ m range and the higher spacing factors (500 to 900 μ m) were obtained without any air entraining agent.

The same mixing procedure was used for all concretes. The superplasticizer was first added to the mixing water, and the cement, the silica fume and the water were mixed to obtain a uniform paste. If an air entraining agent was needed, it was added at this stage, premixed with approximately 200 ml of the mixing water. The coarse and fine aggregates were then placed in the pan-mixer and the concrete was mixed for three minutes.

The required number of 75 x 300 x 400 mm specimens for the scaling tests plus one 75 x 100 x 400 mm prism for the determination of the air void characteristics of the concrete were cast from each batch. Concrete was consolidated in the molds with the use of a vibrating table, and the surface to be tested was simply lightly leveled with a wooden trowel. As previously mentioned, all specimens were cured by sealing them in a number of layers of plastic film for the selected period of 1, 3, 7 or 28 days. Cylinders (100 x 200 mm) were also cast in certain cases for compressive strength measurements.

Deicer salt scaling tests were carried out according to ASTM Standard C 672, following a drying period of 28 days after the curing period. All specimens were covered with a 2.5% sodium chloride solution and subjected to 150 daily cycles. After every 5 cycles, the specimens were weighed and rated as prescribed by the standard (from 0, no scaling to 5, severe scaling) and the sodium chloride solution was renewed. The scaled-off particles were also collected and weighed to allow a better evaluation of the deterioration of the concrete surface. The characteristics of the air void system were determined using the ASTM C-457 "point count" method.

AIR VOID CHARACTERISTICS

The air void characteristics of all mixes are given in Table 4. For each water-cementitious material ratio, the spacing factor varies between low values (100-300 μm), obtained with air entrainment, to high values (500-900 μm), obtained without air entrainment. Intermediate dosages of the air entraining agent were used to obtain values in the 300 μm to 500 μm range, but, in most cases, these median or moderate values are at the lower end of this range.

The air content of the concretes made with no air entraining agent are relatively high, probably because of the high dosage of the superplasticizer which can entrain a significant volume of air. Because of the low water content of the mixes, it was very difficult to produce concretes with high air contents even with a high dosage of the air entraining agent. The low values of the spacing factor were only obtained due to the high specific surface of the air voids.

COMPRESSIVE STRENGTH

Compressive strength tests were carried out at 28 days on a minimum of two concretes for each water-cementitious material ratio. For each of these, the test was made on six 100 x 200 mm cylinders. The results of these tests are given in Table 5 (together with the air content measured on the hardened concrete). The compressive strength varies between 78 MPa and 97 MPa. The highest value (97 MPa) was obtained with the lowest water-cementitious material ratio and no air entrainment.

The variation of the compressive strength with time (from 1 to 28 days) was measured on three additional mixes (again using six 100 x 200 mm cylinders for each measurement). The results of these tests are shown in Figure 1. For all mixes, more than 60% of the 28 day compressive strength was reached after only 24 hours.

SCALING TEST RESULTS

The mass of scaled-off particles after 50, 100 and 150 cycles respectively is given, for each mix tested, in Table 6. All values correspond to the average result obtained from two specimens.

The results in Table 6 indicate no significant scaling after 50 cycles for any of the 27 mixes tested. The highest loss of mass after 50 cycles is only 0.15 kg/m^2 , a very low value which corresponds to a C 672 rating of 0. Even after 150 cycles, the highest loss of mass is still only 0.43 kg/m^2 , a value well below the Norwegian standard limit of 1 kg/m^2 which corresponds to a rating of 2 or 3 (8). In many cases, the small

loss of mass was only due to a very few small and shallow pop-outs.

Figure 2 shows the progression of the scaling process for the least and the most deteriorated specimens for each water-cementitious material ratio. As can be seen in this figure (as well as from Table 6), the deterioration due to salt scaling for all concretes increased slowly and almost linearly during the complete 150 freeze-thaw cycles.

The mass of scaled-off particles for each mix at the end of the 150 cycles is presented in Table 7 in such a way as to put in evidence the influence on the scaling resistance of the water-cementitious material ratio, that of the spacing factor and that of the curing period. It must first be pointed out that all these values are very low, therefore an important part of the difference between the results for the various mixes may be simply associated with the normal variability of such test results.

Table 7 shows that many of the most deteriorated specimens (with a loss of mass $> 0.25 \text{ kg/m}^2$) were obtained from the mixes with a water-cementitious material ratio of 0.30 after the longest curing periods (7 and 28 days). For the other ratios, all values of the loss of mass are extremely low ($< 0.16 \text{ kg/m}^2$), except for two values of 0.25 and 0.23. For each water-cementitious material ratio, the lowest and the highest values were obtained irrespective of the spacing factor values. It can thus be safely said that, considering the general variability of these results and the very small surface deterioration in all cases, the various parameters analyzed in this study did not have any very significant influence on the scaling resistance of the concretes tested.

DISCUSSION

It is well known that in normal concrete the water-cement ratio, the spacing factor and the curing period play a major role with regard to the deicer salt scaling resistance (7,9,10). From the results in Table 7, however, it is clear that this was not the case for the high performance concretes tested in this study, since all of them exhibited a very good performance. This, in fact, is not so surprising since the mechanical and physical properties of normal and high performance concretes are quite different. The use of a Type III Portland cement with 6% silica fume, for instance, allowed the production of a high quality cement paste even after only a very short curing period. With water-cementitious material ratios ranging from 0.30 to 0.23, compressive strength test results show that more than 60% of the 28 day strength (80-100 MPa) was developed after only 24 hours.

The scaling mechanism (or mechanisms) in concrete are very complex and are still not clearly understood (11,12,13). It is known, however, that chloride ions play a major role and it is suspected that the saturation of voids of any type near the

exposed surface can also be important (7). The use of a Type III Portland cement with 6% silica fume in a very low water-cement ratio concrete certainly allowed the production of an extremely dense paste with a minimum porosity and permeability which helped to reduce surface saturation and perhaps reduced the ability of chloride ions to penetrate the cement paste. This, together with the high mechanical strength of the paste, can probably explain why the use of entrained air, which is necessary to protect normal concrete against scaling, was not necessary for the concretes tested in this study.

The fact that it is possible to fabricate high performance concretes which are deicer salt scaling resistant without the use of air entrainment is very important, particularly since the production of a good and stable air void system in low water-cement ratio concrete can be quite difficult and also because of the reduction in compressive strength associated with the use of air entrainment.

Foy et al. (7) have shown that the scaling resistance of a non-air-entrained 0.25 water-cement ratio concrete (without silica fume) after 14 days of water curing can be very good. In this study, similar scaling tests have shown that non-air-entrained concrete with a 0.30 water-cementitious material ratio can be extremely resistant to deicer salt scaling, even after only 24 hours of curing, when a Type III cement with 6% silica fume is used. It is thus clear that extremely low water-cementitious material ratios and long curing periods are not necessary for good scaling resistance in non-air-entrained high performance concrete when appropriate materials are used.

CONCLUSION

The deicer salt scaling resistance of high performance concrete made with Type III Portland cement, 6% silica fume and water-cementitious material ratios ranging from 0.30 to 0.23 was found to be excellent, even after a curing period of only 24 hours. The ASTM C 672 scaling test results show that for these concretes, there seems to be no relationship between the salt scaling resistance the water-cementitious material ratio, on the air void spacing factor, or the length of curing. After 150 cycles, almost no surface deterioration was noted on any of the 27 mixes tested. These results, of course, are only valid for the mixes tested in this study. The excellent durability of these concretes is considered to be due in good part to the low values of the water-cementitious material ratio, but probably also to the use of a Type III Portland cement with 6% silica fume that produces very rapidly an extremely dense et impermeable cement paste.

Additional tests are needed to analyze the exact influence of silica fume and the type of cement on the salt scaling resistance of high performance concrete, particularly at early ages.

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TABLE 1 - Mix Description

W/(C+S) = 0.30		
Mix	Curing period (d)	Spacing factor
30-1-H	1	high
30-3-H	3	high
30-7-H	7	high
30-28-H	28	high
30-7-M	7	moderate
30-28-M	28	moderate
30-1-L	1	low
30-3-L	3	low
30-7-L	7	low
30-28-L	28	low

W/(C+S) = 0.26		
Mix	Curing period (d)	Spacing factor
26-1-H	1	high
26-3-H	3	high
26-7-H	7	high
26-28-H	28	high
26-3-M	3	moderate
26-7-M	7	moderate
26-28-M	28	moderate
26-1-L	1	low
26-3-L	3	low
26-7-L	7	low
26-28-L	28	low

W/(C+S) = 0.23		
Mix	Curing period (d)	Spacing factor
23-3-H	3	high
23-7-H	7	high
23-3-M	3	moderate
23-7-M	7	moderate
23-3-L	3	low
23-7-L	7	low

TABLE - 2 Sand Grading

Sieve size	Percentage passing, cumulative
No. 4 (4.75 mm)	100
No. 8 (2.36 mm)	96
No. 16 (1.18 mm)	88
No. 30 (600 μ m)	58
No. 50 (300 μ m)	24
No. 100 (150 μ m)	7

TABLE 3 - Mix Characteristics

Mix	Water	Cem.	Silica	C.Ag.	F.Ag.	SP	AEA	Slump	Air
	(kg/m ³)					(ml/kg of C+S)		(mm)	(%)
30-1-H 30-3-H	145	457	27	1052	799	28	0	55	2.8
30-7-H 30-28-H	146	455	27	1052	799	25	0	50	2.9
30-7-M 30-28-M	144	450	27	1040	790	32	0.9	65	4.0
30-1-L 30-3-L	143	449	26	1034	785	28	0.9	45	4.4
30-7-L 30-28-L	142	441	26	1019	774	25	1.7	90	4.9
26-1-H	151	548	32	1046	706	38	0	75	2.6
26-3-H 26-7-H 26-28-H	149	546	32	1042	704	38	0	130	2.6
26-3-M 26-7-M 26-28-M	148	546	32	1042	704	38	0.9	90	3.1
26-1-L	145	527	31	1006	680	38	1.9	95	4.9
26-3-L 26-7-L 26-28-L	148	545	32	1038	702	37	1.9	110	3.6
23-3-H 23-7-H	158	651	39	1035	596	52	0	55	2.5
23-3-M 23-7-M	160	645	38	1024	591	60	0.9	40	2.8
23-3-L 23-7-L	156	640	38	1017	586	52	2.8	30	3.9

TABLE 4 - Air Void Characteristics

Mix	Air Content, A		Specific Surface, α (mm^{-1})	Spacing Factor, $\bar{L}$ (μm)
	Fresh (%)	Hardened (%)		
30-1-H 30-3-H	2.8	2.1	9.0	802
30-7-H 30-28-H	2.9	3.0	7.6	817
30-7-M 30-28-M	4.0	3.8	17.4	318
30-1-L 30-3-L	4.4	3.9	22.0	255
30-7-L 30-28-L	4.9	4.6	23.4	216
26-1-H	2.6	1.7	11.3	730
26-3-H 26-7-H 26-28-H	2.6	2.3	9.7	744
26-3-M 26-7-M 26-28-M	3.1	1.5	29.6	302
26-1-L	4.9	4.1	28.1	199
26-3-L 26-7-L 26-28-L	3.6	4.5	19.4	285
23-3-H 23-7-H	2.5	2.8	9.4	778
23-3-M 23-7-M	2.8	3.7	13.9	457
23-3-L 23-7-L	3.9	3.4	39.4	170

TABLE 5 - Compressive Strength

Mix	Air content (%)	Compressive strength (28d) (MPa)
30-1-H 30-3-H	2.1	87
30-1-L 30-3-L	3.9	78
30-7-M 30-28-M	3.8	85
26-3-H 26-7-H 26-28-H	2.3	93
26-3-L 26-7-L 26-28-L	4.5	86
23-3-H 23-7-H	2.8	97
23-3-M 23-7-M	3.7	91

TABLE 6 - Scaling Test Results

Mix	Spacing factor (μm)	Scaled-off particles after 50 cycles (kg/m^2)	Scaled-off particles after 100 cycles (kg/m^2)	Scaled-off particles after 150 cycles (kg/m^2)
30-1-L	255	0.08 (0)	0.13	0.19 (0)
30-1-H	802	0.04 (0)	0.06	0.09 (0)
30-3-L	255	0.04 (0)	0.05	0.10 (0)
30-3-H	802	0.02 (0)	0.03	0.06 (0)
30-7-L	216	0.15 (0)	0.30	0.43 (2)
30-7-M	318	0.08 (0)	0.17	0.25 (1)
30-7-H	817	0.06 (0)	0.11	0.17 (0)
30-28-L	216	0.12 (0)	0.32	0.49 (2)
30-28-M	318	0.05 (0)	0.13	0.17 (0)
30-28-H	817	0.09 (0)	0.20	0.32 (1)
26-1-L	199	0.05 (0)	0.09	0.14 (0)
26-1-H	730	0.02 (0)	0.06	0.08 (0)
26-3-L	285	0.02 (0)	0.03	0.06 (0)
26-3-M	302	0.05 (0)	0.09	0.16 (0)
26-3-H	744	0.05 (0)	0.09	0.12 (0)
26-7-L	285	0.03 (0)	0.04	0.05 (0)
26-7-M	302	0.04 (0)	0.08	0.15 (0)
26-7-H	744	0.05 (0)	0.09	0.11 (0)
26-28-L	285	0.04 (0)	0.08	0.17 (0)
26-28-M	302	0.04 (0)	0.06	0.09 (0)
26-28-H	744	0.03 (0)	0.05	0.07 (0)
23-3-L	170	0.07 (0)	0.13	0.23 (1)
23-3-M	457	0.01 (0)	0.02	0.04 (0)
23-3-H	778	0.05 (0)	0.14	0.25 (1)
23-7-L	170	0.03 (0)	0.08	0.14 (0)
23-7-M	457	0.02 (0)	0.04	0.05 (0)
23-7-H	778	0.02 (0)	0.05	0.08 (0)

(Number in brackets indicates ASTM C 672 visual rating)

TABLE 7 - Mass of Scaled-Off Particles at the End of the 150 Cycles

Mass of scaled-off particles (kg/m ²)												
W/ (C+S)		0.30			0.26			0.23				
Spacing factor		High	Moderate	Low	High	Moderate	Low	High	Moderate	Low		
curing period (d)	1	0.09 (0)	--	0.19 (0)	0.08 (0)	--	0.14 (0)	--	--	--	--	
	3	0.06 (0)	--	0.10 (0)	0.12 (0)	0.16 (0)	0.06 (0)	0.25 (1)	0.04 (0)	0.23 (1)	0.23 (1)	
	7	0.17 (0)	0.25 (1)	0.43 (2)	0.11 (0)	0.15 (0)	0.05 (0)	0.08 (0)	0.05 (0)	0.14 (0)	0.14 (0)	
	28	0.32 (1)	0.17 (0)	0.49 (2)	0.07 (0)	0.09 (0)	0.17 (0)	--	--	--	--	

(Number in brackets indicates ASTM C 672 visual rating)