Effect of the curing conditions of concrete on the behaviour under freeze–thaw cycles*

G. AL-ASSADI[1], **M. J. CASATI**[2], **J. FERNÁNDEZ**[1] and **J.C. GÁLVEZ**[1]

[1] *Departamento de Ingeniería Civil: Construcción, E.T.S. de Ingenieros de Caminos, Canales y Puertos, Universidad Politécnica de Madrid, C/ Profesor Aranguren s/n, 28040 Madrid, Spain,* [2] *Departamento de Vehículos Aeroespaciales, E.U.I.T. Aeronáutica, Universidad Politécnica de Madrid Pl. Cardenal Cisneros s/n, 28040 Madrid, Spain*

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ABSTRACT The aim of this work is to relate the curing conditions of concrete and the addition of an air-entraining admixture with the damage caused by freeze–thaw cycles. In countries with a continental climate, the curing of concrete in summer is performed under climatic conditions of high temperature and low humidity, and during the winter the concrete suffers conditions of freeze–thaw, often accompanied by the use of de-icing salts. This paper shows the experimental results of the behaviour of concrete specimens cured under climatic summer conditions (high temperature and low humidity) and then subjected to freeze–thaw cycles. Curing of the specimens includes conditions of *good* and *bad* practice in relation to wetting and protection of the concrete. It also examines the effectiveness of using an air-entraining admixture in both cases. The experimental programme includes an evaluation of the mechanical properties of the concrete, the study of the cement hydration and the measurement of the volume and pore sizes of the concrete. These tests were performed before and after the application of the freeze–thaw cycles. The results obtained showed that the specimens without air-entraining admixture show a deterioration of mechanical properties after the freeze–thaw test. However, the inclusion of air bubbles benefits the behaviour of concrete against freeze–thaw cycles so even better mechanical properties after the test were observed. This anomalous behaviour is because the cement hydration process continues over the freeze–thaw tests, closing the pore structure. This aspect has been confirmed with the DTA and TG tests performed.

Keywords concrete; freeze–thaw cycles; cement hydration; porosity; curing of concrete.

INTRODUCTION

As the number of publications dealing with frost durability of concrete is far-reaching, this work does not seek to summarise all the contributions to this vast and complex subject. Concrete is a heterogeneous material, composed by aggregates (sand and gravel), cement, water and usually one or several admixtures to improve the fresh and strengthened behaviour. This material behaves well even under adverse weather conditions if it has been adequately designed, cast and cured. In countries with a continental climate concrete is often cast in the summer months, with high temperature and low humidity, with this concrete being subjected to freeze–thaw cycles during winter. This scenario can worsen if de-icing salt is added,[1–3] such us the cases of the slabs and pillars of road bridges.

Good curing process of concrete during the summer is essential for the durability of the material, especially under freezing conditions in the following winters. The microstructure and the hydration grade of the concrete are highly influenced by the curing conditions. In general terms, a good curing process creates a closer porous structure and higher physical and chemical resistance.[4] It should be noted that for almost all types of concrete, resistance to internal cracking due to freezing and thawing cycles can only be achieved if the mixture contains an adequate system of entrained air voids.[3,5]

Several works have clearly indicated that the curing temperature can have a highly significant influence on concrete scaling resistance. But, in the laboratory the curing is generally performed by immersing samples in lime-saturated water or by placing them in a fog room, both cases quite close to saturated conditions. Surprisingly, the number of studies of the influence of the type of curing, combining high temperature and low humidity, on the scaling behaviour of the concrete remains extremely limited. While in certain cases the type of curing is found to have little influence,[6,7] in others, concretes cured with a curing admixture appear to have an improved scaling resistance.[8,9]

Concrete cured under high temperature, such as the steam cured concrete coming from prefabrication plants, modifies the morphology of the hydrated silicates and shows a more crystalline and porous structure.[10–15] This procedure has been found to be detrimental to the scaling resistance of the concrete. However, some authors[16–18] have found an improvement of the scaling resistance of the high-strength concrete when passing from 20 to 60 °C at early ages of the concrete.

The objective of this paper is to study the influence of the curing conditions of the concrete when it is cast under summer conditions (high temperature and low humidity) in the internal deterioration and scaling resistance under freeze–thaw cycles (winter conditions). Conditions of ‘good’ and ‘bad’ practice curing conditions are studied. The effectiveness of using an air-entraining admixture is also studied. The experimental programme includes the evaluation of the mechanical properties of the concrete, study of the cement hydration and measurement of the volume and pore sizes of the concrete. These tests were performed before and after the application of the freeze–thaw cycles.

Whereas the following section examines the experimental programme, Section 3 shows the results. Conclusions are presented in Section 4.

EXPERIMENTAL PROGRAMME

Materials and specimens

Four different concretes were tested, combining two compression strengths (C30 and C45) and the use or not of an air-entrainment admixture. The mixes were cast using the same cement: CEM I 42.5 R. The aggregate consisted of siliceous rolled with grade 0/5 sand and 5/20 crushed limestone gravel. A polycarboxylate superplasticiser was added (0.6% of cement weight). An air-entrained agent was also added to two of the mixtures. The water/cement ratio was 0.5 and 0.4 for the C30 and C45 concretes, respectively. The water content of the admixtures was computed in the w/c ratio. In Table 1, concrete mixes are given.

Table 1 Mix proportions (kg/m^3)

	Concrete C30		Concrete C45	
Component (kg/m^3)	Without air voids	With air voids	Without air voids	With air voids
Cement CEM I 42.5 R	381	381	400	400
Siliceous sand (0/5)	880	880	769	769
Crushed limestone gravel (5/20)	936	936	1167	1167
Water	190	190	160	160
Superplasticiser	2.3	2.3	5.0	5.0
Air-entrainment agent	0	0.2	0	0.2

The concrete was mixed using a vertical-axis planetary mixer with a capacity of 100 l. Fifteen cylindrical specimens of 150ϕ × 300 mm height were cast for each concrete mix.

Concrete curing

All specimens were demoulded after 24 h. They were subsequently stored in a climatic chamber at 30 °C and 37% relative humidity for 28 d. These temperature and humidity values represent the average environmental conditions of summer in central Spain. Under these conditions, two curing processes were performed. The so-called ‘wet curing’ (W) was carried out daily, wetting the specimens inside the climatic chamber during the first week, as recommended by the Spanish Code of Structural Concrete (EHE). The ‘dry curing’ (D) was performed by avoiding the daily wetting of the specimens. The combination of the four types of concrete and the two curing conditions supplied eight different concretes with the following nomenclature: strength of concrete-percentage of air-entrained agent-type of curing. Table 2 shows the combination of these variables and the nomenclature of each cured concrete.

Test procedures and methods

In mechanical terms, three properties were measured: compressive strength (EN 12390–3 standard), elasticity modulus (UNE 83316 standard) and tensile strength (EN 12390–6 standard). The tests were performed before and after the freeze–thaw cycles.

Mercury intrusion porosimetry (MIP) tests were performed with a Micromeritics porosimeter, Autopore IV 9500 model, which reached a pressure of 228 MPa, and measured the diameter of pores from 0.006 to 175 μm.

Table 2 Nomenclature of the types of concrete with different curing process

Strength (MPa)	Air-entrainment agent	Type of curing	Mixture	Designation
30	No	Wet	1	C30–00-W
	Yes		2	C30–0.05-W
	No	Dry	3	C30–00-D
	Yes		4	C30–0.05-D
45	No	Wet	5	C45–00-W
	Yes		6	C45–0.05-W
	No	Dry	7	C45–00-D
	Yes		8	C45–0.05-D

The ASTM D4404 standard was adopted. The weight of the sample was 3.5 ± 0.3 g. The specimen was dried at 40 °C and degasified. For modelling, the pores were idealised as cylindrical channels, and the Washburn–Laplace law was adopted to relate the mercury pressure with the radius of the pore, as Eq. 1 shows

$$p = \frac{-4\gamma \cos\theta}{d}, \tag{1}$$

where γ is the surface tension, θ the contact angle, d the pore diameter and p the pressure required for mercury to penetrate into the pore.

The MIP tests were performed before and after the freeze–thaw cycles. The samples were obtained from cylindrical slices placed at 5 cm from the base of the specimens. In the case of the tests after freeze–thaw cycles, four samples were obtained and tested; these were extracted at different distances from the circumferential face of the specimen in the radial direction. Figure 1 shows a sketch of the specimen with the slices for the various tests.

Thermogravimetric analysis (TG/DTA) was performed to study the hydration of the cement. The STA 791 device was used. The specimen was heated to a temperature of 1000 °C, with a velocity of 10 °C/min. The reference material was α-Al_2O_3, heated to 1200 °C. The tests were carried out in N_2 atmosphere with a flow of 80 ml/min. Results, in accordance with the ASTM E1131 standard, were obtained when the concrete life was at 28, 90 and 365 d.

Freeze–thaw cycle

Six specimens for each type of concrete were tested according to freeze–thaw ASTM C 666 standard.[19] One specimen of each type of concrete was stored at 20 °C and 45–50% relative humidity for comparison.

The test was performed as follows: at the age of 28 d the specimens were immersed in water for four days, ensuring saturation. Then, they were inserted in the climate chamber and were subjected to 300 freeze–thaw cycles according to the temperature versus time diagram shown in Fig. 2.

Every week, i.e. every 42 cycles, the length, weight and ultrasonic pulse velocity were measured in each specimen. The specimens were removed from the climatic chamber for measuring at a temperature of 10 °C. Upon the completion of 300 freeze–thaw cycles, the modulus of elasticity, tensile and compressive strength was measured on each specimen. In accordance with the standard C666 [19] the relative dynamic modulus of elasticity (RDME) was

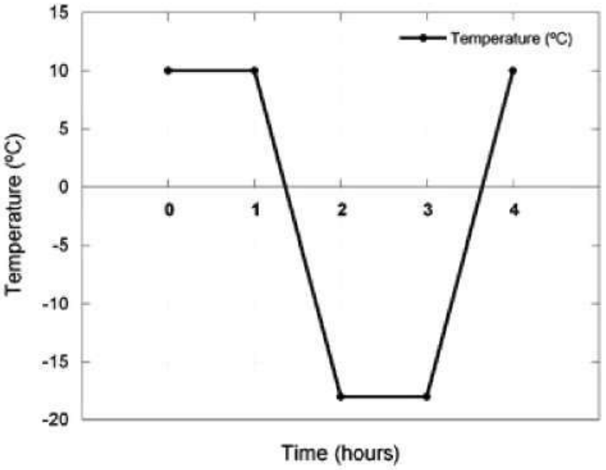

Fig. 2 Freeze–thaw cycle used.

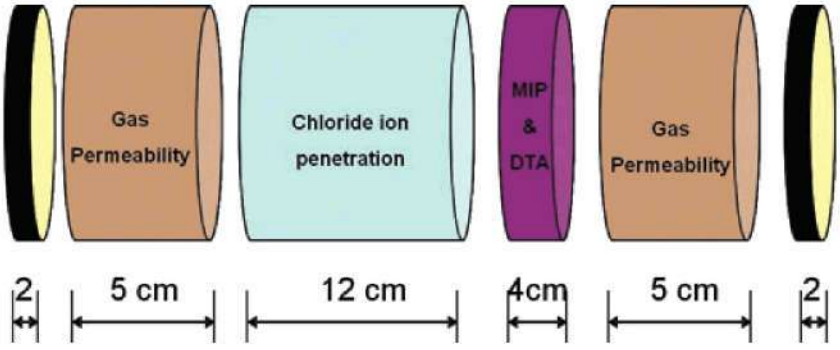

Fig. 1 Sketch of the slices for different tests obtained from a cylindrical specimen.

calculated:

$$P_c = \left(\frac{n_1}{n}\right)^2 \times 100, \tag{2}$$

where: P_c is the RDME, after c freeze–thaw cycles (%), n is the fundamental transverse frequency at 0 freeze–thaw cycles, and n_1 after c cycles.

The durability factor is calculated as follows:

$$DF = \frac{P \times N}{M}, \tag{3}$$

where DF is the durability factor, P is the RDME at N freeze–thaw cycles (%), N is the number of cycles at which P reaches the specified minimum value for discontinuing the test or the specified number of cycles at which the exposure is to be terminated, whichever is less, and M the specified number of cycles at which the exposure is to be terminated.

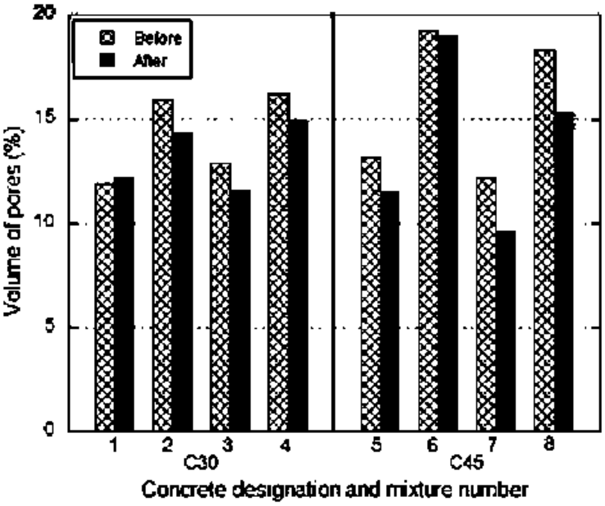

Fig. 3 Volume of pores obtained by MIP for all the concretes before and after 300 freeze–thaw cycles.

RESULTS

Table 3 includes the results for the compressive strength and porosity for the specimens before and after the freeze–thaw cycles.

The C30 specimens without air-entraining agent experienced deterioration in the mechanical properties as a result of the freeze–thaw test; however, this tendency was no clearly observed in the case of the C45 specimens without air-entraining agent. The inclusion of air voids improved the mechanical behaviour of the concrete exposed to freeze–thaw cycles in all cases. The mechanical properties after testing were even improved due to an increase in age (approximately 90 d), in addition to its improved behaviour against the freeze–thaw attack.

Regarding the porosity of the specimens, in almost all cases, a lower porosity was detected after the freeze–thaw cycles, without a significant influence of the air-entraining agent. As can be observed in Fig. 3, in almost all cases, the specimens exposed to moist curing showed a higher porosity after the freeze–thaw cycles than the corresponding specimens exposed to dry curing.

The critical diameter, which refers to the most commonly interconnected pore size, and corresponds to the maximum slope of the accumulated porosity curve, was also evaluated. As can be observed in Fig. 4, it seems that

Table 3 Experimental results of the mechanical properties and MIP tests before and after freeze–thaw cycles

Concrete	Compression strength (MPa)	Modulus of elasticity (GPa)	Tensile strength (MPa)	Mercury intrusion porosimetry: Volume of pores (%)	Mercury intrusion porosimetry: Critical diameter (nm)
C30–00-W (Before)	34.35	31.86	3.91	11.94	50.3635
C30–00-W (After)	21.43	20.85	3.24	12.19	40.2834
C30–0.05-W (Before)	30.86	28.44	3.01	15.98	62.5151
C30–0.05-W(After)	36.96	29.68	3.43	14.33	40.2850
C30–00-D (Before)	40.84	31.75	3.54	12.91	40.2908
C30–00-D (After)	30.01	26.18	3.42	11.62	40.2821
C30–0.05-D (Before)	27.91	29.95	2.74	16.24	40.2951
C30–0.05-D (After)	31.28	32.53	3.52	14.97	40.2851
C45–00-W (Before)	59.13	37.25	4.29	13.18	32.3730
C45–00-W (After)	50.63	37.70	4.93	11.51	32.3710
C45–0.05-W (Before)	35.11	33.31	3.13	19.23	40.2796
C45–0.05-W (After)	39.38	32.18	4.44	19.00	40.2830
C45–00-D (Before)	56.99	37.53	4.27	12.16	62.5305
C45–00-D (After)	63.39	40.68	4.61	9.65	32.3770
C45–0.05-D (Before)	36.34	28.70	3.02	18.31	62.5212
C45–0.05-D (After)	40.64	33.16	4.25	15.30	40.2809

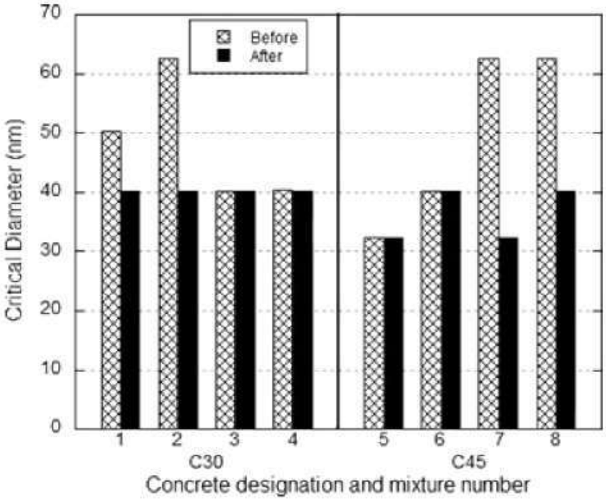

Fig. 4 Critical diameter of pore obtained by MIP for all the concretes before and after 300 freeze–thaw cycles.

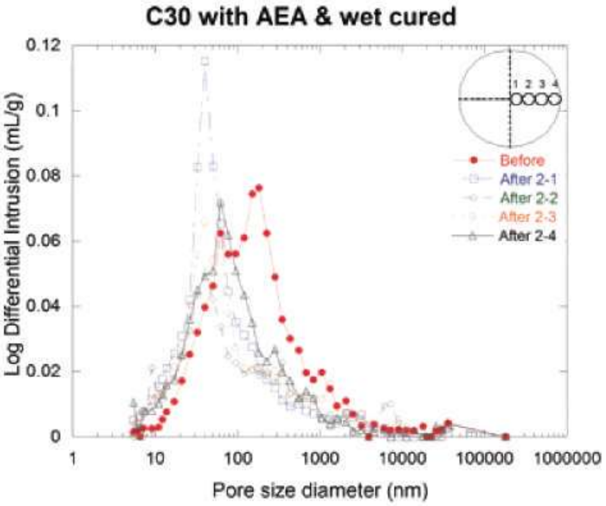

Fig. 6 Distribution of pore diameter for C30–0.05-W before and after freeze–thaw cycles.

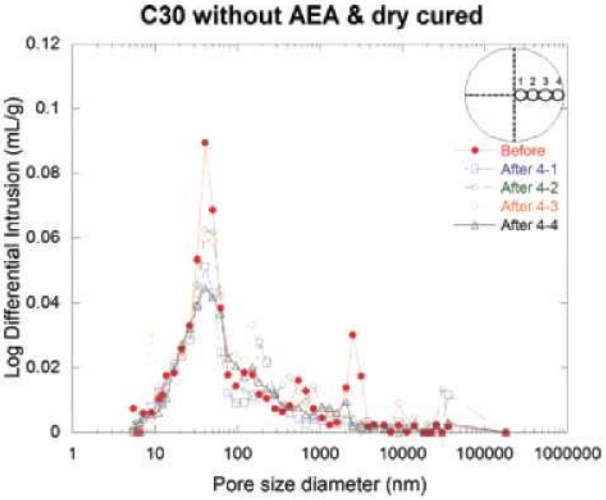

Fig. 7 Distribution of pore diameter for C30–00-D before and after freeze–thaw cycles.

the critical diameter is greater in the case of the specimens exposed to dry curing. This is especially notable in the case of the C45 specimens. After exposure to the freeze–thaw cycles, the critical diameter values were similar for all the specimens.

If the deterioration process caused by the freeze–thaw cycles were the only cause of microstructural variation, the observed pore size distribution after the freeze–thaw cycles would show that the peak of the differential distribution, which represents the dominant pore size, should shift towards the large diameter sizes. As can be observed in Figs 5–12, for the majority of the specimens, and both C30 and C45, this peak does not change. In some cases it even shifts to the left (a smaller pore size), as in the case of the moist cured C30 and dry-cured C45 specimens, indicating a significant increase of the smaller pores after the freeze–thaw cycles. This may be explained by the hydration process stopping, and providing water to the specimens during the freeze–thaw cycles, the hydration process was resumed. This implies that the initial water content of the concrete was not sufficient to avoid self-drying of the specimens, with the re-hydration process being significant in some of the specimens.

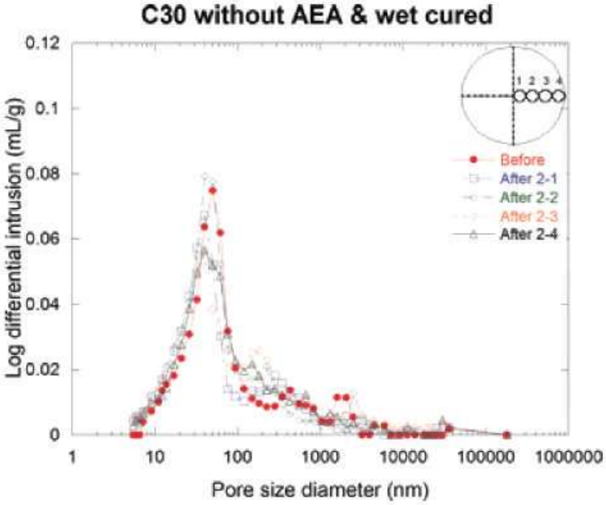

Fig. 5 Distribution of pore diameter for C30–00-W before and after freeze–thaw cycles.

It can also be observed in Table 3 that the total pore volume before the cycles is higher in all specimens, especially in the case of the specimens containing the air-entraining agent. Higher values were observed for the C45 specimens.

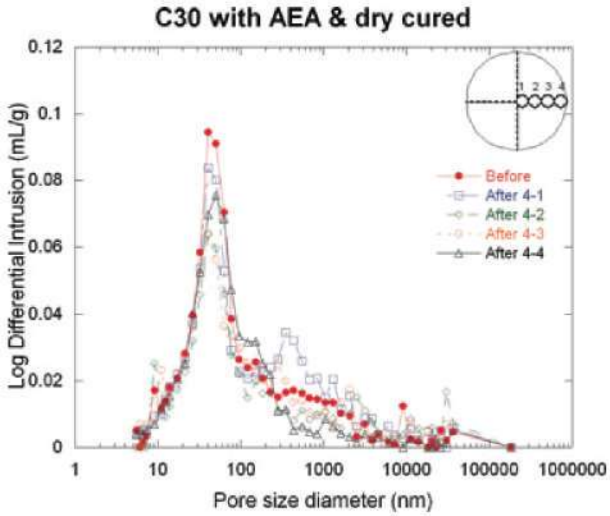

Fig. 8 Distribution of pore diameter for C30–0.05-D before and after freeze–thaw cycles.

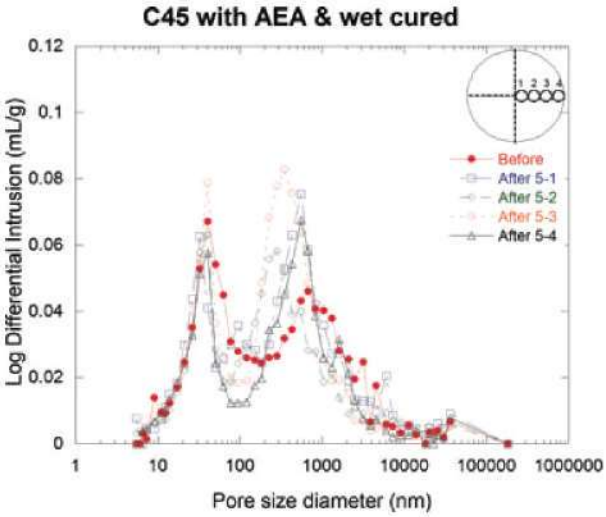

Fig. 10 Distribution of pore diameter for C45–0.05-W before and after freeze–thaw cycles.

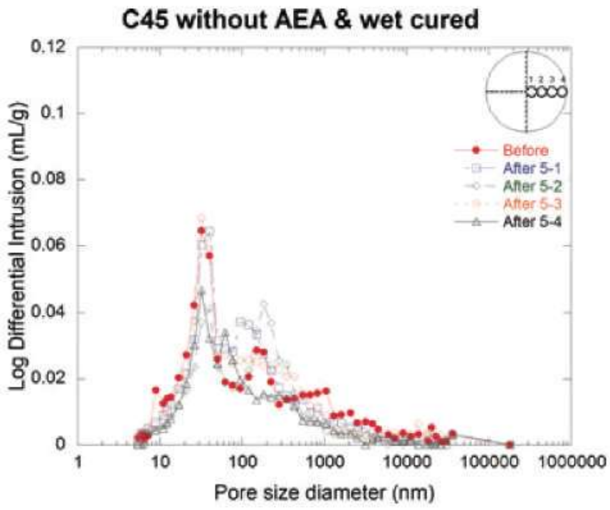

Fig. 9 Distribution of pore diameter for C45–00-W before and after freeze–thaw cycles.

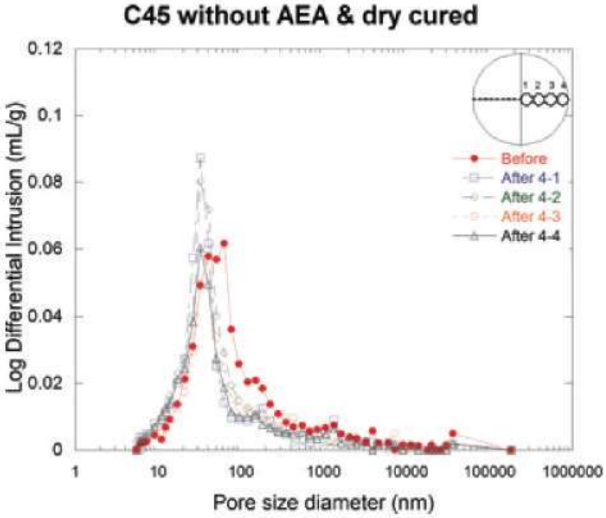

Fig. 11 Distribution of pore diameter for C45–00-D before and after freeze–thaw cycles.

The degree of hydration affects the concrete freezing resistance. A high grade of hydration reduces the capillary porosity of the cement past and, therefore, the quantity of freezing water. Additionally, the strength of the cement paste increases, and consequently there is an increase in the resistance against the hydraulic pressure caused by the freezing of the water.

Figure 13a and b includes the hydration level of the concrete specimens at 28, 90 and 365 d. It can be observed that at 365 d all specimens increase their hydration level. The hydration level reached by the C45 specimens is higher than the C30. This is because of the higher cement content 400 kg/m^3, compared with the 380 kg/m^3 of the C30.

As expected, it was also observed that at 28 d all the dry-cured specimens showed a lower hydration level than the moist cured specimens, with this difference being more notable for the C45 specimens. From these results it can be concluded that the moist-cured specimens experienced a higher hydration rate than the dry-cured specimens. The hydration rate of the dry-cured specimens was slower, resulting in more accessible pore space to the moist movement, and improving the concrete freeze–thaw resistance. This coincides with the higher pore percentage in dry-cured specimens, especially in the case of the C30. This tendency is maintained also at 90 d, though with less pronounced differences. At 365 d, the duration that the specimens were maintained in the climatic

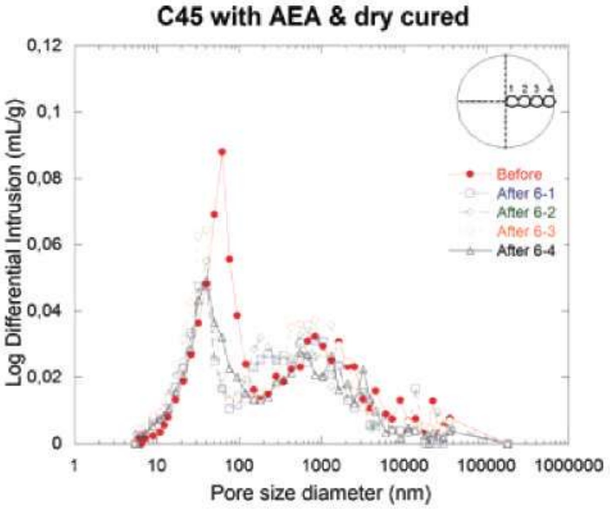

Fig. 12 Distribution of pore diameter for C45–0.05-D before and after freeze–thaw cycles.

chamber, all the specimens cured at extreme conditions reached higher hydration levels than the specimens that were wet during the first week.

This surprising result, according to which concrete under dry-curing conditions improves its behaviour after freeze–thaw cycles, must be treated with caution. The accelerated test in a climatic chamber leads to complete the hydration process of the bad-cured concrete, and this is the reason for the improvement of the concrete behaviour. Obviously, a well-cured concrete with an air-entrainment agent should be the best solution against the freeze–thaw cycles and scaling behaviour of concrete. In the practical case of *in situ* concrete, with a bad curing process, shrinkage cracking will appear and the microstructure will be more porous, with both being detrimental to the freeze–thaw behaviour. The aim of the authors is to emphasise the importance of the real curing process of the *in situ* concrete in combination with the use of an air-entrainment agent; it is apparent that the use or non-use of the air-entrainment agent is the most influential factor affecting the freeze–thaw behaviour of the concrete (see Fig. 14). For these results to be extended to encompass other aspects of durability, such as chloride diffusion when de-icing salts are spread over concrete, complementary studies should be performed.

Figure 15 shows the evolution of the RDME under freeze–thaw testing for the C30 and C45 concretes. In the case of the C30 concrete (Fig. 15a), the specimens without the air-entrainment agent showed worse behaviour, especially the wet-cured specimen.[20] The C45 concrete (Fig. 15b) showed better behaviour, with the behaviour of the specimen without air-entrainment agent and wet cured being a little lower. Figure 16 shows the durability factor of all the concretes after 300 freeze–thaw cycles. These ultrasonic pulse results are consistent with the abovementioned results of MIP and DTA/TG.

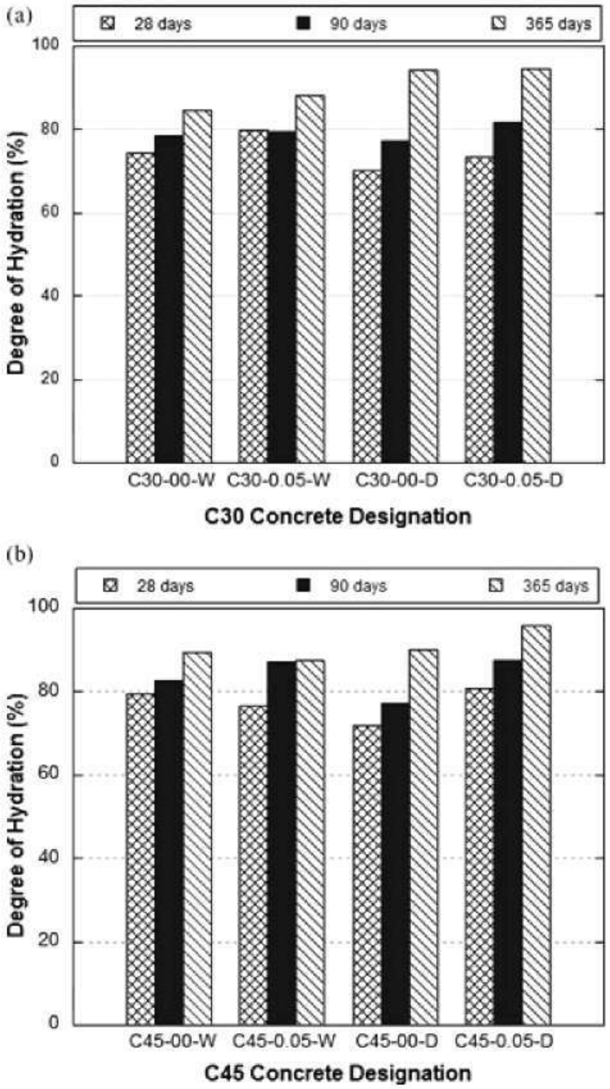

Fig. 13 Degree of hydration of the cement of the concretes: (a) C30, (b) C45.

CONCLUSIONS

The specimens without an air-entraining agent experience generalized deterioration of the mechanical properties as a result of the freeze–thaw test. The specimens with air-entraining agent are more resistant to the freeze–thaw cycles than the specimens without the air-entraining agent. The RDME evolution and Durability Factor seem to be a useful and precise technique to detect the freeze–thawing deterioration at early ages.

The durability characteristics of concrete exposed to freeze–thaw cycles are related to its pore microstructure.

Fig. 14 Specimens C30–0.05-W and C30–00-W after 300 freeze–thaw cycles.

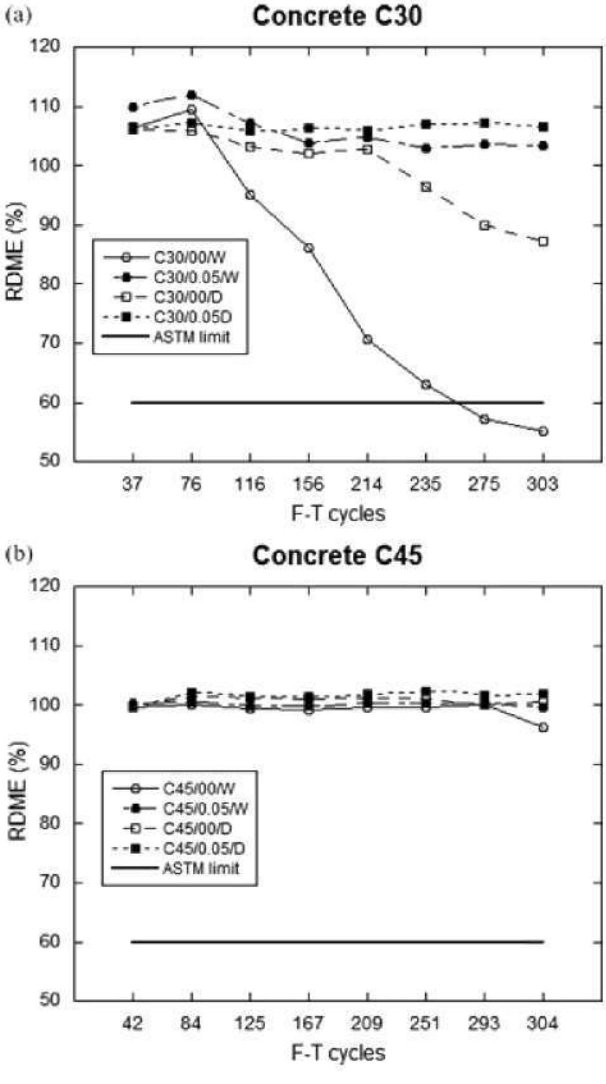

Fig. 15 RDME evolution under freeze–thaw cycles: (a) C30, (b) C45.

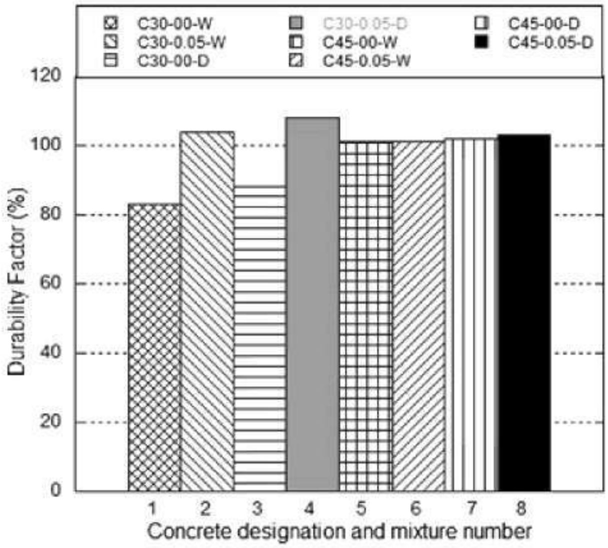

Fig. 16 Durability factor of all concretes after 300 freeze–thaw cycles.

The pore volume, radius and size distribution determine the freezing point of the pore solution and the quantity of ice formed in the pores. It seems that the pore volume and pore size are larger before the freeze–thaw cycles. This is because the hydration of the cement continued during the freeze–thaw test, as it has insufficiently developed during the curing period due to its extreme conditions.

The hydration level of the specimens before the freeze–thaw cycles was approximately 10% lower in the moist-cured specimens and approximately 24% lower in the dry-cured specimens. These results coincide with the pore volume and size results, which were higher before the freeze–thaw cycles. At the end of the cycles, the hydration of the cement, which insufficiently developed during the curing process due to the low humidity and high curing temperature, was completed.

In general, both curing conditions permitted the continuation of the hydration process and the strengthening of the transition zone between the gravel and the paste during the freeze–thaw cycles. The wetting of the specimens further enhanced the hydration process, resulting in a closer pore structure. In contrast, the unwetted specimens resulted in an opener pore structure, with more interconnected pores, which served as beneficial for freeze–thaw resistance.

Considering concrete with water/cement ratio of 0.5 and applied conditions of 37% relative humidity and 30 °C temperature, the type of curing, with or without daily wetting during the first week, do not seem to be significant variables in the deterioration due to freeze–thaw cycles. In certain cases, there was even an improvement in the freeze–thaw resistance of the dry-cured specimens.

The results of the standard C666 [19] should be carefully treated when applied to early-age concretes. Saturated concrete specimens under freeze–thaw cycles may complete the hydration process, leading to erroneous interpretation when extended to the *in situ* concrete, under unsaturated conditions.

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REFERENCES

1 Valenza, J. and Scherer, G. W. (2007) A review of salt scaling: I. Phenomenology. *Cement Concr. Res.*, **37**, 1007–1021.

2 Valenza, J. and Scherer, G. (2007) A review of salt scaling: II. Mechanisms. *Cement Concr. Res.*, **37**, 1022–1034.

3 Pigeon, M., Marchand, J. and Pleau, R. (1996) Frost resistance concrete. *Constr. Build. Mater.*, **10**, 339–348.

4 Auskern, A. B. and Horn, W. H. (1976) Effect of curing conditions on the capillary porosity of hardened portland cement pastes. *J. Am. Ceram. Soc.* **59**, 29–33.

5 Sun, Z. and Scherer, G. W. (2010) Effect of air voids on salt scaling and internal freezing. *Cement Concr. Res.* **40**, 260–270.

6 Klieger, P. and Gebler, S. H. (1987) Fly ash and concrete durability. *ACI Special Publication SP-100* (Edited by J. Scanlon), 1043–1069.

7 Afrani, C. and Rogers, C. (1993) The effect of different cementing materials and curing regimes on the scaling resistance of concrete. In: *Third Canadian Symposium on Cement and Concrete*, Ottawa, Ontario (Canada). pp. 149–166.

8 Bilodeau, A., Carette, G. G., Malhotra, V. M. and Langley, W. S. (1991) Influence of curing and drying on the salt scaling resistance of fly ash concrete. In: *ACI Special Publication SP-126* (Edited by VM Malhotra), In: SP-126 Durability of Concrete: Second International Conference, Montreal, Canada 1991 Farmington Hills, Mich., Durability of Concrete: SP-126. pp. 201–228.

9 Langlois, M., Beaupré D, Pigeon, M. and Foy, C. (1989) The influence of curing on salt scaling resistance of concrete with and without silica fume. *ACI Special Publication SP-114* (Edited by V. M. Malhotra), In: SP-114 Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete: Proceedings of the Third International Conference, Trondheim, Norway, 1989. Editor V. M. Malhotra, Farmington Hills, Mich. 971–990.

10 Neville, A. M. (1995) *Properties of Concrete*. Longman Scientific & Technical. UK.

11 Escalante-García, J. I. and Sharp, J. H. (2001) The microstructure and mechanical properties of blended cements hydrated at various temperatures. *Cement Concr. Res.*, 31 695–702.

12 Kjellsen, K. O., Detwiler, R. J. and Gjorv, O. E. (1991) Development of microstructures in plain cement pastes hydrated at different temperatures. *Cement Concr. Res.*, 21, 179–189.

13 Kjellsen, K. O., Detwiler, R. J. and Gjorv, O. E. (1990) Pore structure of plain cement pastes hydrated at different temperatures. *Cement Concr. Res.*, **20**, 927–933.

14 Price, W. H. (1951) Factors influencing concrete strength. *ACI J.*, 47, 417–432.

15 Copeland, L. E. and Kantro, D. L. (1969) Hydration of Portland cement. In: *Proc. Int. Symp. Chem. Cem.*, Tokyo, pp. 387–421.

16 Khurana, R. and Torresan, I. (1997) New admixtures for eliminating steam curing and its negative effects on durability. In: *ACI Special Publication SP-173*, In: SP-173 Superplasticizers and Other Chemical Admixtures in Concrete. Proceedings. Fifth CANMET/ACI. International Conference. Rome, Italy, 1997. Editor V. M. Malhotra, Farmington Hills, Mich. pp. 83–103.

17 Jacobsen, S., Saether, D. H. and Sellevold, E. J. (1997) Frost testing of high strength concrete: frost/salt scaling at different cooling rates. *Mater. Struct.*, **30**, 33–42.

18 Jonsson, J. A. and Olek, J. (2004) Effect of temperature-match-curing on freeze-thaw and scaling resistance of high-strength concrete. *Cement, Concr. Aggreg.* ASTM International, **26**, 21–25.

19 ASTM C666/C 666M-03 (2008) *Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing*. Vol. 4.02, ASTM International, ASTM C666/C666M - 03(2008) Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing. ASTM International, West Conshohocken, PA. DOI: 10.1520/C0666_C0666M-03. www.astm.org. pp. 341–346.

20 Acebes Pascual, M., Sánchez Martín, T., Al-Assadi and Hernández, M. G. (2008) Caracterización de la durabilidad del hormigón sometido a ciclos hielo deshielo mediante modelos micromecánicos (in Spanish). *Anales de Mecánica de la Fractura*, **25**, 587–592.