ELSEVIER

Cement & Concrete Composites 25 (2003) 409–420

Cement & Concrete Composites

www.elsevier.com/locate/cemconcomp

The durability characteristics of high performance concrete: a review

P.C. Aïtcin *

Faculty of Engineering, Department of Civil Engineering, Université de Sherbrooke, Sherbrooke (Québec), Canada J1K 2R1

Abstract

Durability problems of ordinary concrete can be associated with the severity of the environment and the use of inappropriate high water/binder ratios. High-performance concrete that have a water/binder ratio between 0.30 and 0.40 are usually more durable than ordinary concrete not only because they are less porous, but also because their capillary and pore networks are somewhat disconnected due to the development of self-desiccation. In high-performance concrete (HPC), the penetration of aggressive agents is quite difficult and only superficial. However, self-desiccation can be very harmful if it is not controlled during the early phase of the development of hydration reaction, therefore, HPC must be cured quite differently from ordinary concrete. Field experience in the North Sea and in Canada has shown that HPCs, when they are properly designed and cured, perform satisfactorily in very harsh environments. However, the fire resistance of HPC is not as good as that of ordinary concrete but not as bad as is sometimes written in a few pessimistic reports. Concrete, whatever its type, remains a safe material, from a fire resistance point of view, when compared to other building materials.

Keywords: Curing; Durability; Fire-resistance; Freezing and thawing; High performance concrete

1. Introduction

The recent developments in the field of high-performance concrete (HPC) represent a giant step toward making concrete a high-tech material with enhanced characteristics and durability. These developments have even led to it being a more ecological material in the sense that the components—admixtures, aggregates, and water—are used to their full potential to produce a material with a longer life cycle. Be that as it may, we know that concrete will never be an eternal material when measured against a geological time frame. Any concrete, if we look far enough into the future, will end its life cycle as limestone, clay, and silica sand, which are the most stable mineral forms of calcium, silica, iron, and aluminum in the earth's environment. Therefore, all we can do as engineers or scientists is to extend the life cycle of this artificial rock as much as possible.

The concrete that was known as high-strength concrete in the late 1970s is now referred to as HPC because it has been found to be much more than just stronger: it displays enhanced performances in such areas as durability and abrasion resistance. Although widely used, the expression "HPC" is very often criticized as being too vague, even as having no meaning at all. Since there is no single best definition for the material known as HPC, it is preferable to define it as a low water/binder concrete which receives an adequate water curing.

HPC can be made with cement alone or any combination of cement and mineral components, such as, blast furnace slag, fly ash, silica fume, metakaolin, rice husk ash, and fillers, such as limestone powder. Ternary systems are increasingly used to take advantage of the synergy of some mineral components to improve concrete properties in the fresh and hardened states, and to make high performance concrete more economical and ecological.

It must be emphasized that the development of HPC technology has shown us once more what Féret [1] expressed in his original formula for calculating the compressive strength of a concrete mixture: concrete compressive strength is closely related to the compactness of the hardened matrix.

Fig. 1 represents schematically the fundamental microstructural difference between cement pastes having a 0.65 and 0.25 water/cement ratio. In a 0.25 W/C ratio cement paste, there are more cement grains and

* Tel.: +819-821-7117; fax: +819-821-6949.
E-mail address: pcaitcin@gci.usherb.ca (P.C. Aïtcin).

0958-9465/03/$ - see front matter © 2002 Elsevier Science Ltd. All rights reserved.
doi:10.1016/S0958-9465(02)00081-1

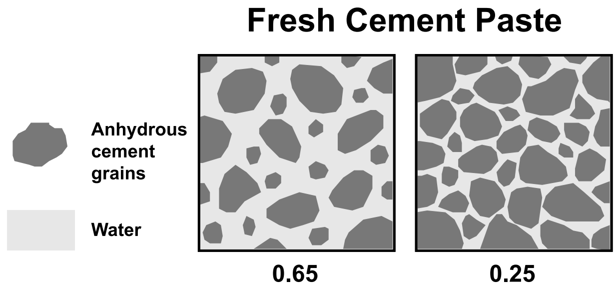

Fig. 1. Schematical representation of the microstructure of two cement pastes having W/C ratios of 0.65 and 0.25.

consequently less water per unit volume so that cement grains are much closer to each other than in a 0.65 W/C cement paste. This major difference results in a completely different type of hydrated cement paste. A 0.65 W/C ratio cement paste is very porous and rich in crystallized outer hydration products formed through a solution–precipitation process, while a 0.25 W/C ratio cement paste is very compact and essentially composed of inner hydration products resembling a gel developed through a diffusion process. Figs. 2 and 3 illustrate the major difference existing between the microstructure of a high and low W/C ratio cement paste. This essential microstructural difference results in a major difference in the mechanical and durability behavior of both the cement paste and the transition zone between the paste and the aggregates.

In particular, in HPC, the coarse aggregate can be the weakest link in concrete when the strength of

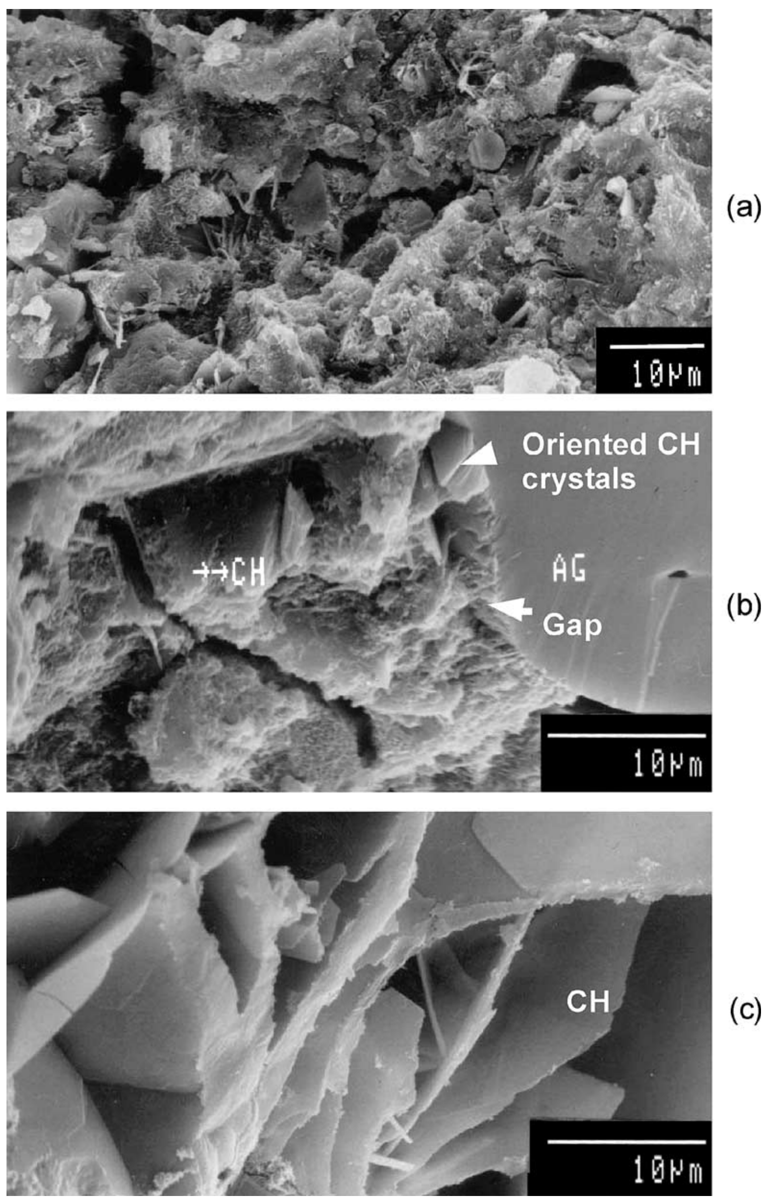

Fig. 2. Microstructure of high water/cement ratio concrete: (a) high porosity and heterogeneity of the matrix, (b) orientated crystal of $Ca(OH)_2$ on aggregate (AG), (c) CH crystals.

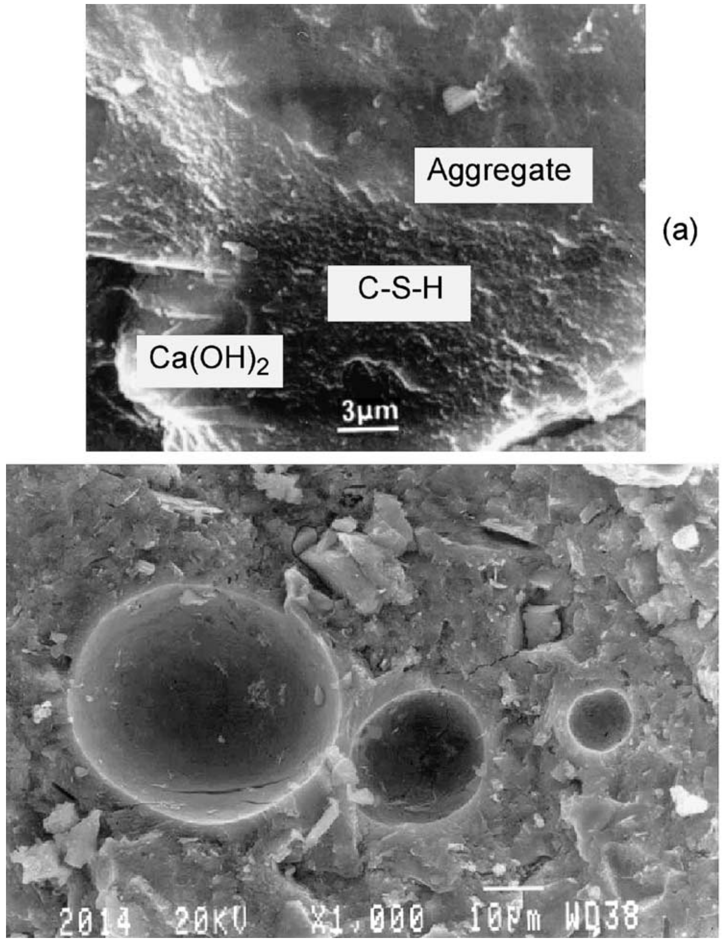

Fig. 3. Microstructure of a HPC: low porosity and homogeneity of the matrix: (a) absence of transition zone between the aggregate and cement paste; (b) dense cement paste in an air entrained high performance concrete.

the hydrated cement paste is drastically increased by lowering its water/binder ratio. In such cases, concrete failure can start to develop within the coarse aggregate. As a consequence, there can be exceptions to the water/binder ratio law when dealing with HPC. In some areas, decreasing the water/binder ratio below a certain level is not practical from a mechanical point of view because the strength of the HPC will not significantly exceed the compressive strength of the aggregate. When the compressive strength is limited by the coarse aggregate, the only way to get higher strength is to use a stronger aggregate. But although the compressive strength is not increased when decreasing the W/B ratio, the compactness of the matrix is increased and the durability of HPC is improved.

2. Volumetric changes

As with any other material, the volume of concrete changes as its temperature changes. Like any other material concrete creeps. But it is not the only volumetric variations exerting itself on concrete. Depending on its curing condition, concrete presents volumetric variations, it usually shrinks but sometimes it swells. In this paper, swelling of chemical origin, such as sulfate or thaumasite attack or alkali aggregate reaction will not be considered, the only volumetric variation taken into account will be plastic shrinkage, autogenous or isothermal shrinkage, and drying shrinkage [2]. Carbonation shrinkage will not be considered because it is a very slow process that takes place much later.

In all cases that will be considered in this paper, the origin of the volumetric variation is the same, the appearance of tensile stresses in the menisci created in the fresh concrete as it is drying (plastic shrinkage) or in the hardened concrete due to self-desiccation (autogenous shrinkage) and due to drying (drying shrinkage).

Autogenous shrinkage is a consequence of the chemical contraction occurring in the cement paste when water hydrates cement particles. In fact, the absolute volume of the hydrates formed is smaller than the sum of the absolute volume of the cement particles and the water that have reacted. Hydration creates some 8% voids, as found by Le Chatelier and Powers [3]. This very fine porosity drains water from the coarser capillaries where water is not as strongly bonded. Consequently, as hydration progresses it is observed that the coarse capillaries are being emptied (as in the case of drying shrinkage) but without any mass loss. This phenomenon is called self-desiccation. Self-desiccation is due to the movement of the water that is moving from the preexisting coarse capillaries towards the very fine porosity created by cement hydration.

Drying shrinkage occurs when concrete dries in dry air, as concrete looses some of its internal water; menisci appear within the coarse superficial capillaries. In the case of drying shrinkage there is a mass loss.

In ordinary concrete with W/C ratio greater than 0.50, for example, there is more water than required to fully hydrate the cement particles and a large amount of this water is contained in well connected large capillaries so that the menisci created by self-desiccation appear in large capillaries where they generate only very low tensile stresses. Therefore, the hydrated cement paste barely shrinks when self-desiccation develops (40–60 microstrains) [4].

In the case of HPC with a W/B ratio of 0.35 or less, significantly more cement and less mixing water have been used, so that the initial pore network is essentially composed of very fine capillaries. When self-desiccation starts to develop, as soon as hydration begins, menisci rapidly develop into small capillaries if no external water is added. Since many cement grains start to hydrate simultaneously in HPC, the drying of the very fine capillaries can generate high tensile stresses that shrink the hydrated cement paste. This early shrinkage is referred to as autogenous shrinkage. Of course, autogenous shrinkage is as large as drying shrinkage observed in ordinary concrete when these two types of drying develop in capillaries of the same diameter [2].

But, when there is an external supply of water, the capillaries do not dry out as long as they are connected to this external source of water [5]. The result is that no menisci, no tensile stress, and no autogenous shrinkage develop within a HPC thin element having a W/C ratio of 0.35 that is constantly water cured from the moment of its setting. But when the W/C ratio is lower than 0.35 or at the center of a large concrete element made with a 0.35 W/C ratio HPC, concrete microstructure can be so dense that water penetration can be stopped and self-desiccation can develop in certain parts of concrete. In fact, when cement particles are hydrating with water coming from an external source there is an increase in the absolute volume of the cement that leads to the filling of some pores and capillaries. In this case, it would be more appropriate to speak of isothermal shrinkage rather than autogenous, since autogenous shrinkage refers to the shrinkage of a closed system.

Thus, the essential difference between ordinary concrete and HPC is that ordinary concrete exhibits practically no autogenous shrinkage, whether it is water cured or not, whereas HPC can experience significant autogenous shrinkage if it is not water cured during the hydration process. Autogenous shrinkage does not develop in HPC as long as the pores and capillaries are interconnected and have access to external water, but, when the continuity of the pore and capillary systems is broken, then, and only then does autogenous shrinkage start to develop within the hydrated cement paste of a HPC, as shown in Fig. 4.

Drying shrinkage of the hydrated cement paste begins at the surface of the concrete and progresses more or less rapidly through the concrete, depending on the relative humidity of the ambient air and the size of capillaries. Drying shrinkage of ordinary concrete is therefore rapid because the capillary network is well connected and contains open capillaries at the surface of the concrete. Drying shrinkage in HPC is slow because capillaries are very fine and soon get disconnected.

Another major difference between drying shrinkage and autogenous shrinkage is that drying shrinkage develops from the surface inwards, while autogenous shrinkage is homogeneous and isotropic, insofar as the cement particles and water are well dispersed within the concrete.

Thus, there are considerable differences between ordinary concrete and HPC with respect to their shrinkage behavior. The cement paste of an ordinary concrete exhibits rapid drying shrinkage progressing from the surface inwards, whereas HPC cement paste can develop a significant isotropic autogenous shrinkage when not water cured. This difference in the shrinkage behavior of the cement paste has very important consequences for concrete curing and concrete durability.

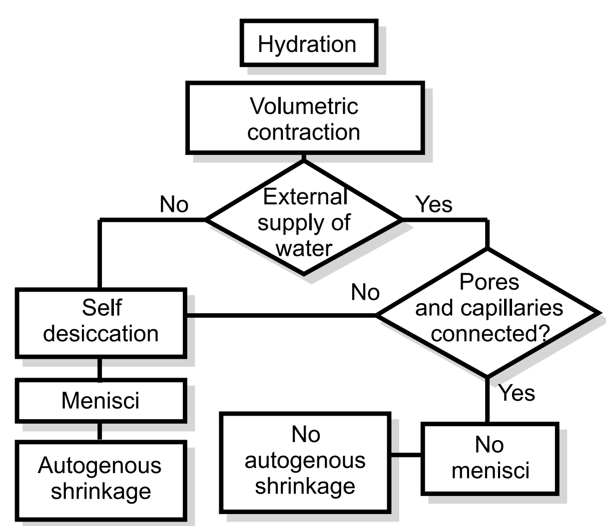

Fig. 4. Influence of curing conditions on the occurence of autogenous shrinkage.

Although the shrinkage of a hydrated cement paste is a very important parameter with respect to concrete volumetric stability, it is not the only one. A key parameter is the amount of aggregate, and, more specifically, the quantity of coarse aggregate. Too often it is forgotten that aggregates do more than simply act as fillers in concrete. In fact, they actively participate in the volumetric stability of concrete when they restrain the shrinkage of the hydrated cement paste: concrete shrinkage is always much lower than that of a cement paste having the same W/C ratio. It is common knowledge that concrete shrinkage can be easily reduced by increasing the coarse aggregate content; but it must not be forgotten that the shrinkage of the hydrated cement paste stays the same, it is simply more restrained and there is less cement paste, so that the volumetric stability of the concrete is increased. Restraining the shrinkage of hydrated cement paste by modifying the coarse aggregate skeleton may or may not produce a network of microcracks, depending on the intensity of the tensile stresses developed by this process with respect to the tensile strength of the hydrated cement paste.

3. Curing concrete

HPC must be cured quite differently from ordinary concrete because of the difference in shrinkage behavior described above, as emphasized in Fig. 5. If HPC is not water cured immediately following placement or finishing, it is prone to develop severe plastic shrinkage because it is not protected by bleed water, and later on develops severe autogenous shrinkage due to its rapid hydration. While curing membranes provide adequate protection to ordinary concrete (which is insensitive to autogenous shrinkage), they can only help prevent the development of plastic shrinkage in HPC but have no value in inhibiting autogenous shrinkage.

The critical curing period for any HPC runs from placement or finishing, up to 2 or 3 days later, and the most critical period is usually between 12 and 36 h. In fact, the short time during which efficient water curing must be applied to HPC can be considered a significant advantage over ordinary concrete.

Those who specify and use HPC must be aware of the dramatic consequences of missing early water curing. Initiating water curing after 24 h is too late, because most of the time, a great deal of plastic and autogenous shrinkage have already occurred and, by this time, the capillary and pore network are disconnected in many places and the microstructure is already so compact that external water has little chance of penetrating very deep into the concrete.

Water ponding or fogging is the best way to cure HPC; one of these two methods must be applied as soon as possible, immediately following placement or finishing. An evaporation retarder can be applied temporarily to prevent the development of plastic shrinkage. If, for

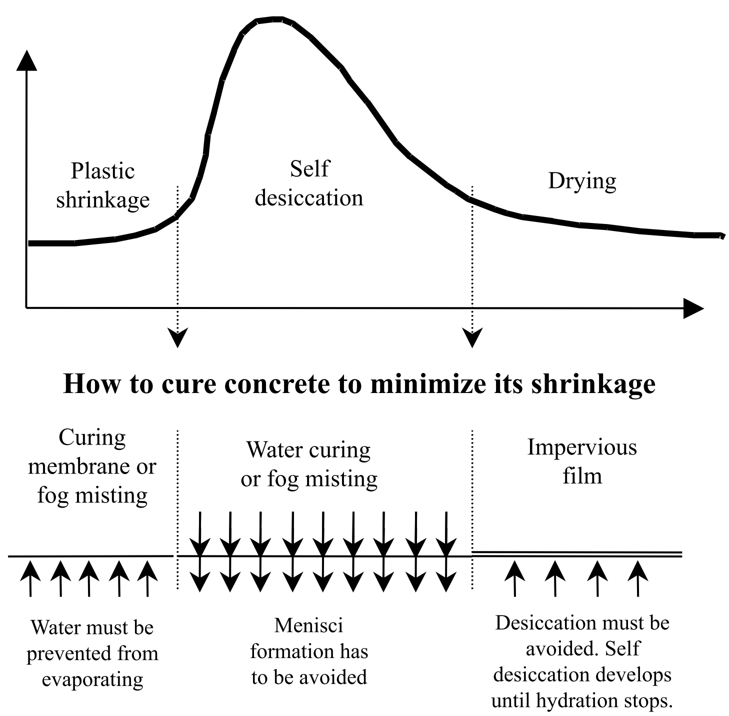

Fig. 5. The most appropriate curing regimes during the course of the hydration reaction.

any reason, water ponding or fogging cannot be implemented for 7 days, then the concrete surface should be covered with wet burlap (hessian) or preferably a prewetted geotextile. The burlap or the geotextile must be kept constantly wet with a soaker hose and protected from drying by a polyethylene sheet in order to ensure that at no time during the curing period is the concrete allowed to dry and experience any autogenous shrinkage [6].

Moreover, it is observed that when any concrete is water cured during setting it does not shrink but rather swell. Fig. 6 illustrates the effect of early water curing on the volumetric change of concrete.

Water curing can be stopped after 7 days because most of the cement at the surface of concrete has hydrated and any further water curing has little effect on the development of shrinkage. After 7 days of water curing, HPC experiences slow drying shrinkage due to the compactness of its microstructure, and that autogenous shrinkage has already dried out the coarse capillaries pores. Even then, theoretically the best thing to do is to paint HPC or to use a sealing agent so that the last water that remains in concrete can be retained to contribute to hydration. There is no real advantage of painting or sealing a very porous concrete because it is impossible to obtain an absolutely waterproof coating; painting or sealing HPC, however, can be easy and effective.

Partial replacement of coarse aggregate by an equivalent volume of saturated lightweight aggregate has been used to counteract autogenous shrinkage internally [7]. The saturated lightweight aggregate particles act as small water reservoirs throughout the mass of concrete; they can fill the very fine pores created by hydration reactions. Therefore, the water of the lightweight aggregate particles is drained along with that contained in the fine capillaries of the HPC. The menisci within the cement paste are not developed in small capillaries, which means lower tensile stress and lower autogenous shrinkage. Lightweight aggregate also reduces compressive strength and elastic modulus. Shrinkage reducing admixture can also be used [8].

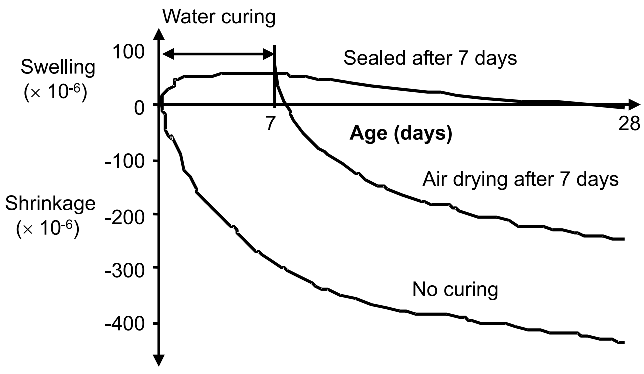

Fig. 6. Length changes according to different curing regimes for the 0.35 W/C ratio concrete.

Another very interesting curing concept has been recently proposed by Jensen and Hansen [9]. It consists of mixing concrete with a super absorbant polymer, of the type found in baby diapers. This super absorbant polymer becomes saturated with water and therefore later on can act as a water reservoir in the hardening concrete. This very clever way of providing an internal source of water, well distributed within the concrete, has lead to the new concept of "water entrained concrete".

It is well known that concrete is never cured properly in the field, despite the fact that it is always written in the specifications that contractors have to cure concrete. Contractors are not curing concrete for a very simple reason: they are not specifically paid for it, therefore, concrete curing is always perceived by them as an unprofitable activity or even a source of expense and therefore a waste of time. But, when contractors are specifically paid to water cure concrete they do it as they would for any other item that is paid for. For three years now, the City of Montreal and the Québec Ministry of Transportation have requested unit prices for each item directly related to early water curing. Since the initiation of this new policy on the early water curing of concrete, it is amazing to see how zealous contractors can become in the matter of water curing. For them water curing is now seen as a source of profit. From the first experiences in that matter it has been found that the cost of an early water curing is about one tenth of 1%, a very modest price when considering the improved durability of the concrete structures that are built this way.

Therefore, the best way to be sure that HPCs are properly and efficiently cured in the field is to specifically pay contractors to cure concrete [6].

This very long introductory remarks were made to emphasize that two important key parameters control the penetration of any aggressive agents in concrete: the water/cement or the water/binder ratio, and the curing of concrete. Specifying a low water/binder ratio concrete is a necessary condition, but not a sufficient one.

4. Durability

4.1. General matters

The durability of a material in a particular environment can only be established over time [10], so it is difficult to precisely predict the longevity of HPC due to lack of a track record for HPC exposed to very harsh environments for more than 5–10 years, except perhaps for some North Sea offshore platforms, which have been in operation for more than 25 years.

It must be remembered that the first uses of HPC in the late sixties and early seventies were indoor applications, mainly in columns in high-rise buildings, which are not subjected to a particularly severe environment. Outdoor applications of HPC only date back from the late eighties and early nineties, which means that not enough time has gone to properly assess the real service life of HPC structures under outdoor conditions. But, based on the experience with ordinary concrete, we can safely assume that HPC is more durable than ordinary concrete. Indeed, the experience gained with ordinary concrete has taught us that concrete durability is mainly governed by concrete permeability and the harshness of the environment [11].

It is easy to assess the harshness of any environment with respect to HPC because hydrated cement paste is essentially a porous and basic material that contains some freezable water. This assessment simply involves examining how the environment affects each of these characteristics. On the other hand, it is not always simple to assess how easily aggressive agents will penetrate concrete. For example, the flow of water through a 0.70 W/C concrete is easy to measure, but water flow almost stops in a 0.40 W/C ratio concrete, regardless of the thickness of the sample and the amount of pressure applied. The gas permeability is also difficult to measure and sample preparation, particularly drying, significantly influences the measured value of gas permeability. Therefore, the critical question remains how to appropriately assess the permeability of a concrete with a low W/B ratio and a very compact microstructure.

Despite all the criticisms, the so-called "Rapid chloride-ion permeability test" (AASHTO T-277) gives a fair idea of the interconnectivity of the fine pores in concrete that are too fine to allow water flow. Chloride-ion permeability is expressed in Coulombs, which corresponds to the total amount of electrical charge that passes during the 6-h test through a concrete sample subjected to a difference of potential of 50 V.

When the rapid chloride-ion permeability test is performed on concrete samples with low W/C ratio, the number of Coulombs passing through the sample decreases significantly. It is easy to achieve a chloride-ion permeability of less than 1000 C for a HPC containing about 10% silica fume and having a W/B ratio around 0.40–0.45. The only other way to achieve this would be with latex-modified concrete, which would be much more costly. Much lower chloride-ion permeability values can be achieved if the W/B ratio is reduced below 0.25. Values as low as 150 C have been reported, far lower than the 5000–6000 C reported for ordinary concrete [12]. The rapid chloride-ion test also reveals that the connectivity of the pore system decreases drastically as the W/B ratio decreases, making the migration of aggressive ions or gas more difficult in HPC than in its plain counterpart. The author believes that this is the best indication that the service life of HPC should exceed that of ordinary concrete in the same environment. It is difficult to determine the number of years by which the service life would be extended because the predictive models developed for ordinary concrete cannot be readily extrapolated to include HPC. However, it can be said that some HPC structures will outlast the average life span of a human being.

4.2. Durability in a marine environment

4.2.1. Nature of the aggressive action

Seawater by itself is not a particularly harsh environment for plain concrete, but a marine environment can be very harmful to reinforced concrete due to the multiplicity of aggression that it can face [13]. In a marine environment, a concrete structure is essentially subjected to four types of aggressive factors:

- chemical factors related to the presence of various ions dissolved in the sea water or transported in the wet air;
- geometrical factors related to the fluctuation of the sea level (waves, tides, storms, etc.);
- physical factors such as freezing and thawing, wetting and drying, etc.;
- mechanical factors such as the kinetic action of the waves, the erosion caused by sand in suspension in the sea water, floating debris and even floating ice in northern seas.

It is the combination of these different factors that can be harmful to reinforced concrete structures. In the following sections a brief review of the nature of each attack will be presented to show how high performance concrete is best fitted to resist not only each of these particular factors, but also their combined action.

4.2.2. Chemical attack on concrete

As previously stated, seawater is not particularly harmful to plain concrete, several submerged plain concrete blocks and structures exposed for many years in different marine environments are still in a relatively good condition. The only chemical limitation usually recommended for a cement to be used in a marine environment is related to its C_3A content, which should not be greater than 8%. Fig. 7 represents the different successive altered zones found in a concrete structure exposed to sea water for several years: carbonation, formation of brucite and monochloroaluminate, and sulfate attack with the formation of gypsum, ettringite, or even thaumasite. Each of these chemical mechanisms is well known and explained in specialized books [3,13–15].

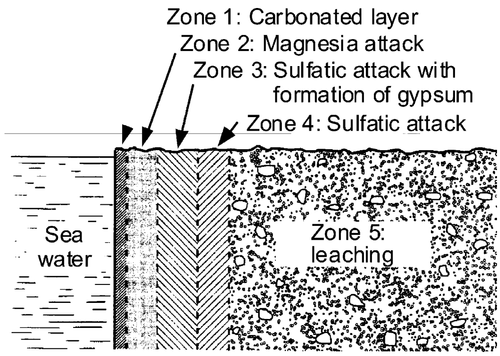

Fig. 7. Schematic representation of the different altered layers found in a concrete marine structure.

Sea water can be very harmful to reinforced concrete because, once chloride ions have reached reinforcing steels, it results in a rapid spalling of the cover concrete, and consequently it is easier for chloride ions to reach the second level of reinforcing steel, and so on. The only way to inhibit, or to retard as long as possible the corrosion of the steel by chloride ions is:

- to specify a very dense and impervious concrete, and place and *cure* it correctly, and;
- to increase the concrete cover.

The development of all these mechanisms of aggression is closely related to the facility with which aggressive ions can penetrate concrete; therefore it is obvious that a very dense and impervious matrix, like the one found in HPC, constitutes the best protection that can be presently offered against a marine environment.

HPC has been used very successfully for more than 20 years to build offshore platforms in the North Sea, and more recently to build two major bridges for which the owner had requested a 100-years service life, one of these bridges is the Confederation Bridge in Canada and the other is the Tago Bridge in Lisbon, Portugal. It is interesting to point out that these two bridges have been built in a build, own, operate and transfer (BOOT) mode by consortia of contractors that will have to maintain these two bridges during the entire concession period. In that respect, it is interesting to note that in the case of these two bridges, the concrete cover has been extended to 75 mm to meet the 100-year life cycle requirement.

It is also very important to point out that it is not sufficient to specify a Type V or slag cement to obtain a concrete that will resist the harsh marine environment. The curing of this concrete is as important as the selection of an appropriate cementitious system.

4.2.3. Abrasion resistance

In this case also, the denseness and high resistance of the matrix of a HPC offers a good protection to the abrasive action of the sand, of the debris, or even from floating ice. In the case of the Confederation Bridge, the concrete used to build the conical part of the piles that deflects the ice loads in the tidal zone was a 90 MPa air-entrained HPC. This concrete is thought to be able to resist the tidal freezing and thawing cycles in winter and the abrasive action of the floating ice which is particularly severe in the Northumberland Straight, due to the presence of changing currents associated with the tides and winds.

4.3. Freeze–thaw resistance

There is still controversy about the necessity of entraining air in HPC to make it freeze–thaw resistant [16]. In Canada, any exposed HPC must be air entrained, which is the case of the concrete of the Confederation Bridge [17]; in Norway, HPC can contain a small amount of air, but more to facilitate its placing and finishing than to improve its freeze–thaw resistance.

This subject has always been and remains controversial. First of all, no single test can be used to ascertain if a particular concrete is resistant to freezing and thawing. Standards such as ASTM C 666, propose more than one procedure for determining freeze–thaw resistance, and selecting the proper procedure is not always straightforward. Secondly, the freezing and thawing rate can vary over a large range when these tests are performed, and the variation in rate can influence test results. Thirdly, an arbitrary value for the durability factor is usually specified to distinguish a freeze–thaw resistant concrete from one that is not. Finally, there is the issue of how many freeze–thaw cycles a concrete must resist in order to be declared freeze–thaw resistant.

In North America, freeze–thaw resistance of concrete is assessed using Procedure A (freezing and thawing in water) of ASTM C 666. If the durability factor of concrete is still above 60% after 300 cycles, the concrete is said to be freeze–thaw resistant. Because this test takes too long to perform (usually more than 10 weeks) several other criteria giving a more rapid assessment of freeze–thaw resistance have been developed and correlated with the ASTM C 666 test. This is the case, for example, with the use of the spacing factor as a freeze–thaw acceptance criterion. Measuring the spacing factor of a particular concrete is not so easy, but it can be done within a week or less. To illustrate this point, the present Canadian Standard CSA A23.1 specifies that ordinary concrete can be classified as freeze–thaw resistant if its average spacing factor is less than 230 μm with no individual values higher than 260 μm. When the CSA Committee adopted this criterion, it was noted that these values also protected ordinary concrete from the scaling action of deicing salts. This fact has generally been forgotten.

Experience has proven that this criterion is too severe for HPC. HPCs with a spacing factor as high as 350 μm, and even 425 μm in one case, was found to resist 500 freeze–thaw cycles. Therefore, the spacing factor has been slightly increased and the 2000 formulation of CSA A23.1 now states that when a HPC has a W/B ratio lower than 0.36 its spacing factor has to be lower than 250 μm with no individual value more than 300 μm in order to be considered freeze–thaw resistant.

It is still not clear how many cycles HPC should withstand before it is considered freeze–thaw resistant. A recent study carried out by Aïtcin [17] involving different HPCs with the same W/B, but with spacing factors varying from 190 to 425 μm, revealed an inverse relation between the spacing factor and the number of freeze–thaw cycles to failure. For instance, it took 2000 freeze–thaw cycles to fail a 0.35 W/B ratio HPC with a spacing factor of 190 μm when tested in accordance with ASTM C 666 procedure A.

Two HPC mixtures were used to reconstruct two entrances to a MacDonald's restaurant in Sherbrooke [18]. The non-air-entrained concrete used to build one entrance failed to meet both CSA A23.1 and ASTM C 666 criteria; the air-entrained concrete passed ASTM C 666, but failed CSA A23.1 criteria. After eight winters, during which it can be assumed that the concretes averaged 50 freeze–thaw cycles annually in "saturated conditions" with exposure to deicing salts, the difference between the behavior of the two concretes is still imperceptible.

5. Fire resistance of HPC

For many years, the fire resistance of HPC has been also a controversial subject, some reports saying that HPC performed as well as ordinary concrete, others the reverse [19–26]. Following the first fire that occurred in a HPC structure, the channel tunnel fire [27,28], and from different studies in progress in several countries, it is clear that the fire resistance of HPC does not seem to be as good as that of ordinary concrete, but that it is not as bad as some alarming reports presented it. As is the case for any concrete, HPC is one of the safest construction materials as far as fire resistance is concerned.

As construction details, as well as material resistance by itself could influence greatly the fire resistance of a structural element, it is presently impossible to give very simple rules that should govern the design of HPC structures that are expected to be exposed to a more or less severe fire. Presently, some models are being developed that could be used to forecast the structural consequence of a fire on the safety of a HPC structure. Moreover, several promising avenues are presently being investigated to improve the fire resistance of HPC itself and it must not be forgotten that for inside applications, 50 mm of gypsum is still the best and most economical way to protect any material from fire.

Instead of reviewing in detail the controversial literature on the fire resistance of HPC, three brief presentations on the actual fire resistance of HPC and ordinary concrete will be done. The first one will deal with the violent fire that occurred in the Channel tunnel, based on a report by Demorieux [28], the second one will deal with the big conflagration that occurred in Düsseldorf airport [29], and the third one will present the latest findings of the Brite–Euram HITECO BE-1158 research project.

5.1. The channel tunnel fire

The fire of a truck in the channel tunnel did not surprise the safety department of the channel administration. The occurrence of such a fire had been forecast because each day a truck burns in France and another in England [27]. If a fire was to happen in the channel tunnel, the engine-man of the train was asked to speed up to get the train out of the tunnel as soon as possible.

It had also been forecast that some hydraulic jacks used to support the access ramp used by the trucks to get on the railway platform could be loosened and create a risk of derailment for the train. In such a case, the engine man would be asked to stop the train and tighten the hydraulic jack. But what had not been forecast was that the two incidents could happen simultaneously so that the engine-man could receive two conflicting orders at the same time for which no priority had been included in the safety procedures.

Faced with two contradictory orders the engine-man decided on November 11, 1996 to stop the train 18 km from the entrance to France, fortunately in the driest zone of the Channel tunnel from a water seepage point of view. In that area, the blue chalk through which the tunnel was excavated was the most impervious of all the rock formations.

It is difficult to imagine the damage that could have occurred to the channel tunnel if the fire had taken place a few kilometers further in a fault area where it would not have been possible to take advantage of the imperviousness of the chalk layer.

The fire was particularly violent as far as its maximum temperature and length are concerned. The maximum temperature reached was in the order of 700–1000 °C and the total duration of the fire was about 10 h.

According to Demorieux [28], a standard ISO 834 fire test would have caused a 30 mm thick spalling in the lining of the channel tunnel.

The concrete lining in the area of the fire was composed of precast concrete elements having a design strength of 70–80 MPa and a water/binder ratio of 0.32 [28]. In the most intense zone, the concrete lining was severely damaged and would not have been able to counteract the hydrostatic pressure for which it has been designed. An extensive survey done after the fire on the unaffected zone has shown that the in-place concrete had an average compressive strength comprised between 70 and 80 MPa, a modulus of rupture between 7 and 8 MPa, and an elastic modulus between 37 and 44 GPa.

It was observed that in this zone the reinforcing steel arrangement played a very important role because the concrete, which was prevented from falling down by the reinforcing bars, protected the concrete laying behind. In many places in the central zone, the reinforcing steel was highly deformed due to the severity of the fire. Moreover, under the effect of the heat, the restrained dilatation of the lining generated large transversal and horizontal stresses in the damaged concrete so that numerous 45° fissures could be observed at the edges of the precast elements. Concrete was literally spalled at the centre of the reinforcing mesh over a more or less thick depth depending on its location from the centre of the fire.

Concrete spalled in small pieces of an average thickness of about 10 mm. Some of these pieces were as small as a coin. The concrete lining was always more damaged in its upper zone than at its bottom end on the track.

In the two less damaged zones, concrete had spalled over greater areas in some places. It was possible to see that it happened frequently where nylon spacers had been used during precasting to correctly place the reinforcing steel. Most probably the pressure of the gas generated during the burning of the nylon spacers was responsible for this spalling.

All the tests done in the non-damaged part of the lining in the fire zone have shown that the residual concrete remaining in the lining was almost intact in the upper part as well as in the lower part of the section. SEM observations have shown that the residual concrete had not been altered significantly and no permanent strains could be seen even at the surface of the residual concrete, in all cases the residual concrete was altered on a very thin layer. Finally, according to petrographic examinations, it was concluded that the maximum temperature reached by the concrete in that area was lower than 700 °C.

The observations and test conducted on the concrete of the track and at the lower part of the lining resulted in the decision to keep them in place during reconstruction.

Based on all the observations done and the results obtained, it was decided to rebuild the lining using a wet shotcrete after a careful cleaning of the damaged concrete.

5.2. The Düsseldorf airport fire

On April 16th, 1996 a devastating conflagration broke out at the Düsseldorf airport, killing 17 people by smoke inhalation and injuring several hundred. The fire was attributable to improper use of combustible insulating material and the plastic cover of the cables laid in the hollow ceiling space [29]. The fire was triggered by some welding work which had been performed, and developed unnoticed for a long time so that the smoke had time to spread into all parts of the building in the hollow ceiling and through the ventilation system.

The highest estimated temperature to which the concrete ceiling was exposed has also been estimated to be 1000 °C. At such a temperature, the ordinary concrete spalled, but from a structural point of view the building elements were not damaged. Moreover, it was found that harmful substances such as dioxins did not penetrate into the concrete so that the concrete structure was still in a serviceable condition. But since owners and operators intended to erect an extended and modernized airport facility, they decided to demolish the burnt concrete structure.

5.3. Spalling of concrete under fire conditions

It is difficult to make a direct comparison between these two major conflagrations but it can be pointed out that in both cases, very fortuitously, the initial cause of the fire was the burning of some polystyrene, that the maximum temperature reached has been estimated at 1000 °C, and that the HPC of the channel tunnel and the ordinary concrete of Düsseldorf Airport both spalled.

Of course, the thickness of the spalled concrete is a function of the maximum temperature that was reached during the fire and of the duration of exposure to this temperature, as it has been seen in the various zones of the fire area in the channel tunnel.

5.4. The Brite–Euram HITECO BE-1158 research project

The preliminary conclusions of this research project, financed by the European Community, were presented on March 9th, 1999 at a meeting of the French Civil Engineering Association meeting. All the tests were done in Finland at the V.T.T. Laboratories, one of the best-equipped European laboratories for fire studies [30].

The conclusion of this presentation is: the experimental study undertaken on a 60 MPa HPC without silica fume and a 90 MPa HPC with silica fume has shown an excellent fire resistance, except for a small column that was very heavily loaded.

In actual structures more favorable conditions are found because of the following:

- the columns have a bigger size than the one tested;
- the loading is the service loading and not the maximum loading; and
- FIREXPO software can be used to predict the thermo-mechanical behavior of any structural element.

6. Concluding remarks

HPC is not a passing fad. It is here to stay, not only because of its high strength, but also because of its durability.

Therefore, at the dawn of the 21st century, it is not difficult to anticipate that the use of HPC will increase in order to extend the service life of concrete structures exposed to severe environments [11]. The durability of concrete structures depends on several factors, one of which is the durability of the concrete itself. As the durability of concrete is essentially linked to its permeability, HPC, with its dense microstructure and very low permeability, should obviously be more durable than ordinary concrete.

It must be emphasized, however, that good construction practice, including good curing, is essential to produce a durable structure. It would be a pity if improper placing practice and poor curing resulted in a structure with impervious concrete having many cracks. As we still do not know how to make HPC with low permeability, but without high strength, designers have to learn to take advantage of the extra strength provided by low W/B ratio concrete. One day, we may be able to make durable concrete of low strength.

References

[1] Féret R. Sur la compacité des mortiers hydrauliques, Mémoire et documents relatifs à l'art des constructions et au service de l'ingénieur. Annales des Ponts et Chaussées 1892;4(2^{e} semestre): 5–161.

[2] Aïtcin P-C, Neville AM, Acker P. Integrated view of shrinkage deformation. Concr Int 1997;19(9):35–41.

[3] Neville AM. Properties of Concrete. 4th ed. London: Longham; 1995.

[4] Davis HE. Autogenous volume change of concrete. In: Proceedings of the ASTM 43rd Annual Meeting, Atlantic City, NJ, June. 1940. p. 1103–13.

[5] Aïtcin P-C. Non-shrinking concrete. In: Supplementary Papers, CANMET/ACI/JCI 4th International Conference on Recent Advances in Concrete Technology, Tokushima, Japan, June. 1998. p. 215–26.

[6] Standard Specification for High-Performance Concrete, City of Montreal, HPC 93VM-20, Canada, 1998, 19.

[7] Weber S, Reinhardt H. Improved durability of high-strength concrete due to autogenous curing. In: Fourth CANMET/ACI International Conference on Durability of Concrete. SP-170. ACI Publication; 1997. p. 93–121.

[8] Shah SP, Weiss WJ, Yang W. Shrinkage cracking. Can it be prevented?. Concr Int 1998;20(4):31–7.

[9] Jensen OH, Hansen PF. Water-entrained cement-based materials. 1.- Principe and theoretical background. Cement Concr Res October 2000;31(4):647–54.

[10] Mehta PK. Durability of concrete—fifty years of progress. In: Durability of Concrete: Second International Conference, Montreal, Canada 1991. ACI SP-126; 1991. p. 1–132.

[11] Aïtcin P-C. Durable concrete—current practice and future trends. In: Mehta PK, editor. Concrete technology: past, present, and future. ACI SP-144; 1994. p. 83–104.

[12] Gagné R, Lamothe P, Aïtcin P-C. Chloride-ion permeability of different concretes. In: Proceedings of the Sixth International Conference on Durability of Building Materials Components, Omiya, Japan. 1993. p. 1171–80.

[13] Duval R, Hornain H. La durabilité du béton vis-à-vis des eaux agressives. In: Baron J, Ollivier J-P, editors. La durabilité des bétons, Presses de l'École Nationale des Ponts et Chaussées, Paris. 1992. p. 376–85.

[14] Aïtcin P-C. High performance concrete. London: E&F SPON; 1998.

[15] Mehta PK, Monteiro P. Concrete—microstructure, properties, and materials. New York: McGraw-Hill; 1993.

[16] Hammer TA, Sellevold EJ. Frost resistance of high strength concrete. In: High-Strength Concrete: Second International Symposium. ACI SP-121; 1990. p. 457–88.

[17] Aïtcin P-C, Pigeon M, Pleau R, Gagné R. Freezing and thawing durability of high performance concrete. In: Proceedings of the International Symposium on High-Performance Concrete and Reactive Powder Concretes, Sherbrooke '98, vol. 4. 1998. p. 383–91.

[18] Lessard M, Dallaire É, Blouin D, Aïtcin P-C. High-performance concrete speeds reconstruction of McDonald's. Concr Int 1994; 16(9):47–50.

[19] Diederich U, Spitzner J, Sandvik M, Kepp B, Gillen M. The behavior of high-strength lightweight aggregate concrete at elevated temperature. In: High strength concrete. 1993. p. 1046–53.

[20] Noumowe AN, Clastres P, Delvicki G, Costaz J-L. Thermal stresses and water vapour pressure of high-performance concrete at high temperature. In: Proceedings of Utilization of high strength/high performance concrete, Presses de l'École Nationale des Ponts et Chaussées, Paris. 1996. p. 561–70.

[21] Sanjayan G, Stocks LJ. Spalling of high-strength silica fume concrete in fire. ACI Mater J 1993;90(2):170–3.

[22] Chan SYN, Peng GF, Chan JKW. Comparison between high strength concrete and normal strength concrete subjected to high temperature. Mater Struct 1996;29:616–9.

[23] Khoylou N, England GL. The effect of elevated temperature on the moisture migration and spalling behaviour of high strength and normal concretes. In: High-strength concrete: an international perspective. ACI SP-167; 1996. p. 263–89.

[24] Breitenbücker R. High strength concrete C105 with increased fire resistance due to propylene fibers. In: Proceedings of Utilization of high strength/high performance concrete, Presses de l'École Nationale des Ponts et Chaussées, Paris. 1996. p. 571–8.

[25] Jensen BC, Aarup B. Fire resistance of fibre reinforced silica fume based concrete. In: Proceedings of Utilization of high strength/high performance concrete, Presses de l'École Nationale des Ponts et Chaussées, Paris. 1996. p. 551–60.

[26] Phan LT. Fire performance of High-Strength Concrete: A Report of the State-of-the-Art, Res Rep NISTIR 5934, NIST, Gaithersburg, Maryland, USA, 1997.

[27] Acker P, Ulm F-J, Levy M. Fire in the channel Tunnel—Mechanical Analysis of the Concrete Damage, Concrete Canada. In: Proceedings of the Technology Transfer Day, October 1, Toronto, Ontario. 1997 (17 p).

[28] Demorieux J-M. Le comportement des BHP à hautes températures—État de la question et résultats expérimentaux. In: 'Ecole Française du Béton et le Projet National BHP 2000, November 24–25, Cachan, France. 1998 (27 p).
[29] Neck U. Personal communication. March 1999.
[30] Cheyrezy M, Beloul M. Comportement des BHP au feu. In: Proceedings of the French Civil Engineering Association Meeting, March 9. 1999 (7 p).