5 High Performance / High Strength Concretes

HPC / HSC

High performance concrete HPC structures would show not only environmental benefits, but also the cost efficiency. Moreover, HP material properties (higher ductility, fire safety, water tightness, frost resistance, etc.) make structures more durable and more resistant against climatic effects and safer in case of exceptional loads (like climatic disasters or explosions).

There is a big potential for the use of HPC to form low-concrete-volume structures with reduction of the use of primary raw materials, and correspondent reduction of associated environmental impacts.

Materials to produce HPC/HSC

1. **Cementitious materials**

Portland cement—Portland cement PC is by far the most widely used type of cement in the manufacture of concrete, and HSC is no exception. The choice of PC for HSC is extremely important.

A reliable estimate of cement performance in HSC can be achieved by assessing the cements' normal consistency and setting times along with cube strength.

The effect of cementitious material characteristics on water demand is more pronounced in HSCs because of higher cementitious materials contents and low water-cementitious material ratios (w/cm).

The type and amount of cementitious materials in an HSC mixture can have a significant effect on temperature development within the concrete.

Supplementary cementitious materials—fly ash, silica fume, natural pozzolans and slag cement, are now covered under the term "supplementary cementitious materials" (SCMs). SCMs for use in concrete are materials that have mineral oxides like those found in PC, but in different proportions. SCMs are widely used in the production of HPC/HSC because their presence alters the mineral constituents in the binding (paste) system to allow attainment of high strengths.

Evaluation and selection—Cementitious materials, like any material in a HSC mixture, should be evaluated using laboratory trial batches to establish optimum desirable qualities.

2. **Chemical admixtures**

Retarding chemical admixtures (ASTM C494/ C494M, Types B and D)—HPC/HSC mixtures incorporate higher cementitious materials contents than normal-strength concrete. Retarding chemical admixtures are highly beneficial in controlling early hydration, particularly as it relates to strength.

Normal-setting chemical admixtures (ASTM C494/C494M, Type A)—Type A water-reducing chemical admixtures, can provide strength increases while having minimal effect on rates of hardening.

High-range water-reducing chemical admixtures (ASTM C494/C494M, Types F and G)—One potential advantage of HRWRAs is decreasing the w/cm and providing high-strength performance, particularly at early (24-hour) ages. HRWRA may serve the purpose of increasing strength through a reduction in the w/cm while maintaining equal slump, increasing slump while maintaining equal w/cm, or a combination thereof.

Chemical admixture combinations—Combining HRWRAs with water-reducing or retarding chemical admixtures has become common practice to achieve optimum performance at lowest cost.

3. Aggregates

Production of HPC/HSC requires purposeful selection of quality aggregates. Both fine and coarse aggregates used should, as a minimum, meet the requirements of ASTM C33.

Fine aggregate—Fine aggregates with a rounded particle shape and smooth texture have been found to require less mixing water in concrete; for this reason, they are preferable in HPC/HSC. Sand with a fineness modulus of approximately 3.0 gave the best workability and compressive strength.

Coarse aggregate—Coarse aggregate (CA) mineralogical characteristics, grading, shape, surface texture, elastic modulus (stiffness), and cleanliness can influence concrete properties.

In HSC, CA volumes typically range between 50 and 70%. For optimum compressive strength with high cementitious material contents and low w/cm, the maximum size of CA should be kept to a minimum, at 1/2 or 3/8 in. (13 or 10 mm). CA with a rough surface texture is generally more suitable for use in HSC than CA with a smooth surface texture because of the superior bond that it provides.

CONCRETE MIXTURE PROPORTIONS

Concrete mixture proportions for HPC/HSC have varied widely. Factors influencing mixture proportions include the strength level required, test age, material characteristics, and type of application.

ACI 318—As with most structural concretes, HSC is usually specified in terms of its compressive strength. ACI 318 specifies concrete strength requirements. Structural concrete is normally proportioned so that the average compressive strength test results exceed specified strength f'_c by an amount sufficiently high to minimize the frequency of test results below the specified compressive strength.

Testing Age

Twenty-eight days—A common test age for compressive strength of normal-strength concrete is 28 days. Performance of structures has been empirically correlated with the strength of moist-cured concrete cylinders, usually (150 × 300 mm) or (100 × 200 mm). This has produced good results for normal-strength concretes not requiring early strength or early evaluation.

Later age—HSCs made with SCMs may gain considerable strength at later ages and, therefore, are typically evaluated at later ages, such as **56 or 90 days**. HSC has been placed frequently in columns or shear walls of high-rise buildings. Therefore, it has been desirable to take advantage of long-term strength gains so that efficient use of construction materials is achieved.

Estimating compressive strength—

The compressive strength that a concrete will develop at a given w/cm or (w/b) depends on the cementitious materials, aggregates, and admixtures employed. Principal causes of variations in compressive strengths at a given w/cm (w/b) include the strength-producing capabilities of the cement and the hydraulic or pozzolanic activity of SCMs, if used.

The table below gives broad guidelines on f'_c range of HPC corresponding to water/binder ratios

W/B	0.40 – 0.35	0.35 – 0.30	0.30 – 0.25	0.25 – 0.20
Max f'_c range (MPa)	50 – 75	75 – 100	100 – 125	> 125

Estimating compressive strength with time

Concrete strength at any time $(f'_c)_t$ can be estimated from the following general equation:

$$(f_c')t = \frac{t}{\alpha + \beta t}(f_c')_{28}$$

Where α in days and β are constants. $(f'_c)_{28}$ = 28-day strength. t is the age of concrete in days.

The range of α is (0.05 to 9.25) and for β is (0.67 to 0.98). α and β depend on cement type and curing type. This equation is applicable for both NWC and LWC.
Typical values of $\alpha = 4$ and $\beta = 0.85$ are recommended for <u>moist cured</u> concrete of type I OPC.
Typical values of $\alpha = 1$ and $\beta = 0.95$ are recommended for <u>steam cured</u> concrete of type I OPC.

The ratio of $[(f'_c)_t / (f'_c)_{28}]$ for these values are given in the following table:

Time Ratio	Type of Curing	Cement Type	Constants a, β and a/β	Concrete Age: Days 3	7	14	21	28	56	91	Years 1	10	Ultimate (in time)
$(f'_c)_t/(f'_c)_{28}$ Eq. (2-1)	Moist Cured	I	a = 4.0, β = .85	**.46**	**.70**	**.88**	**.96**	**1.0**	**1.08**	**1.12**	**1.16**	**1.17**	**1.18**
		III	a = 2.3, β = .92	.59	.80	.92	.97	1.0	1.04	1.06	1.08	1.09	1.09
	Steam Cured	I	a = 1.0, β = .95	.78	.91	.98	1.0	1.0	1.03	1.04	1.05	1.05	1.05
		III	a = .70, β = .98	.82	.93	.97	.99	1.0	1.0	1.01	1.01	1.02	1.02

Example: Estimate $(f'_c)_{42}$ and $(f'_c)_{56}$ for moist cured concrete of OPC type I if $(f'_c)_{28}$ **= 28 MPa**

$(f'_c)_{42} = [42 / (4 + 0.85×42)] (28) = (1.058) (28) = 29.6$ MPa
$(f'_c)_{56} = [56 / (4 + 0.85×56)] (28) = (1.085) (28) = 30.4$ MPa

PROPERTIES OF HIGH-STRENGTH CONCRETE

For HSC, the uniaxial compressive strength is usually much higher than 56 MPa. Thus, the compressive strength is often obtained by using (100 × 200 mm) cylinders rather than (150 × 300 mm) because of the capacity limitation of testing machines. The difference may vary from 1 to 5%. ACI 363 indicates (100×200 mm) cylinders are suitable for acceptance testing purposes.

Stress-strain behavior in uniaxial compression

Axial stress-versus-strain curves for concrete of compressive strength up to 14,000 psi (98 MPa) are shown in Fig. 1. The shape of the ascending part of the stress-strain curve is more linear and steeper for HSC, and the strain at the maximum stress is slightly higher for HSC. The slope of the descending part becomes steeper for HSC compared with normal strength concrete.

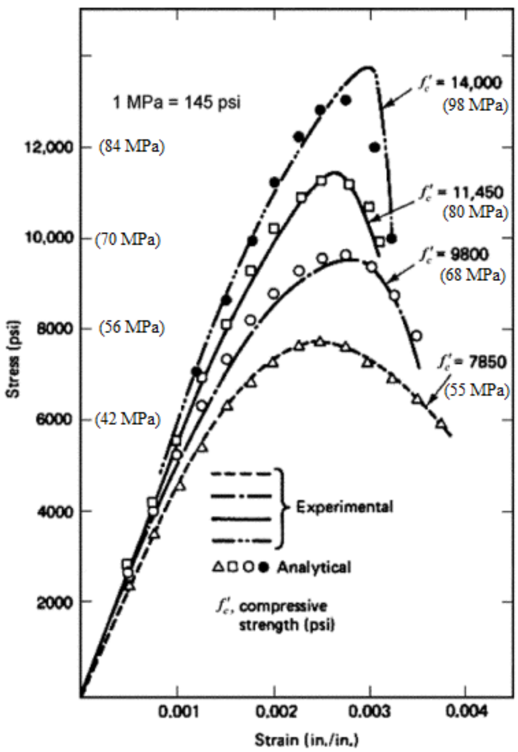

Fig. 1: Stress-strain curves for HSC

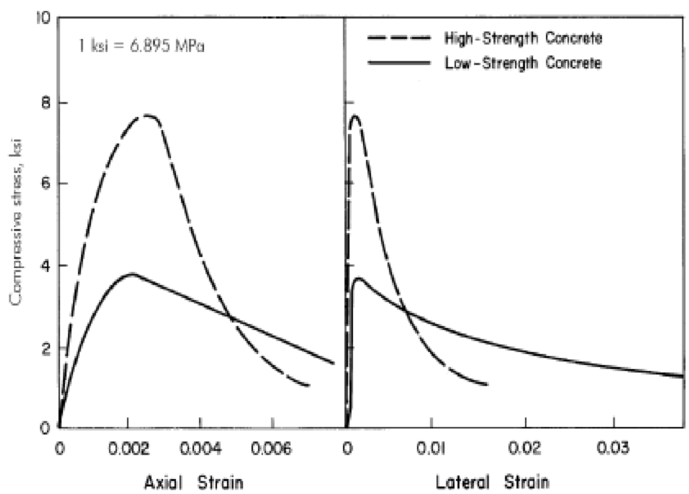

Fig. 2: Axial stress vs. axial strain and lateral strain

HSC exhibits less internal microcracking than lower-strength concrete for a given imposed axial strain. As a result, the relative increase in lateral strains is less for HSC. (Fig. 2)

Modulus of Elasticity

A comparison of several reported empirical equations including the expression given in ACI 318-19, for a concrete density of 145 lb/ft^3 (2350 kg/m^3) is presented in Fig. 3. No single empirical expression estimates the modulus of elasticity for concretes with compressive strengths over 8 ksi (56 MPa) to a high degree of accuracy for the data set.

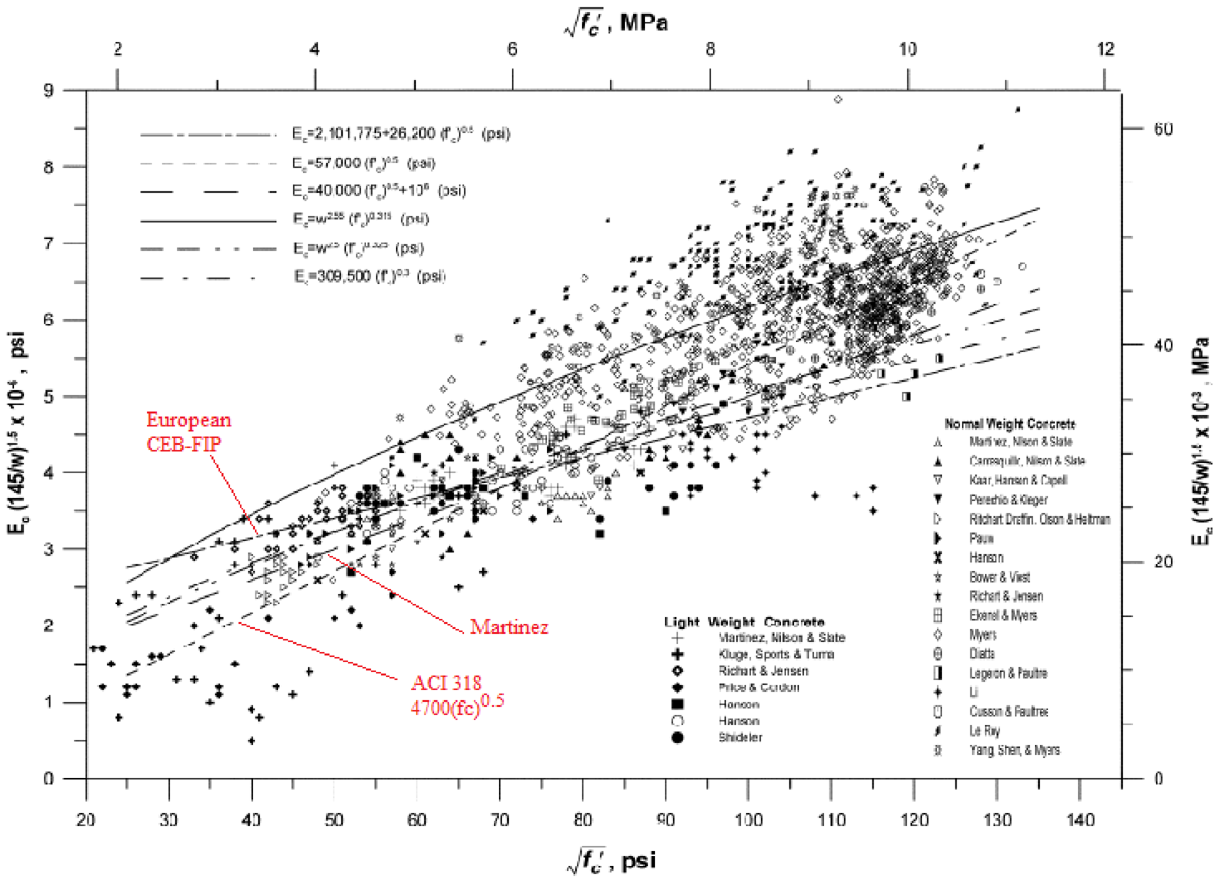

Fig. 3: Modulus of elasticity vs $\sqrt{f_c'}$

A correlation between the modulus of elasticity $\boldsymbol{E_c}$ and the compressive strength $\boldsymbol{f_c'}$ for normal-density concretes has been reported by several researchers:

Equation below **MARTINEZ EQ.** has generally proven to be a relatively reliable lower-bound expression (Fig. 3) for normal-density HSC based on most HSC data collected

$$\boldsymbol{E_c\ (\text{MPa}) = 3320(f_c')^{0.5} + 6900} \qquad \text{for } 21 \text{ MPa} < f_c' < 83 \text{ MPa}$$

[MARTINEZ EQ.]

The European "**CEB-FIP Model Code 1990**" relates the modulus of elasticity to the cube root of the compressive strength rather than the square root (for $f_c' < 80$ MPa):

$$\boldsymbol{E_c\ (\text{MPa}) = 21{,}500\ \alpha_\beta\ [(f_{ck} + 8) / 10]^{1/3}} \quad \textbf{or} \quad \boldsymbol{= 21{,}500\ \alpha_\beta\ [f_{cm} / 10]^{1/3}}$$

where $\boldsymbol{f_{ck}}$ is the characteristic compressive strength of (150 × 300 mm) cylinder; $\boldsymbol{f_{cm}}$ is the compressive strength at 28 days; and $\boldsymbol{\alpha_\beta}$ is a variable for the aggregate type:

$\boldsymbol{\alpha_\beta}$ =1.2 for basalt, dense limestone aggregates,
= 1.0 for quarzitic aggregates,
= 0.9 for limestone aggregates,
= 0.7 for sandstone aggregates.

Poisson's ratio

Poisson's ratio of HSC in the elastic range seems comparable to the expected range of values for lower-strength concretes. Poisson's ratio for HSC having uniaxial compressive strengths (56 to 84 MPa) at 28 days found to be (**0.20** to **0.28)**.

Modulus of Rupture

For NSC, ACI Code recommends $\boldsymbol{f_r}$ **(MPa) = 0.62** $\boldsymbol{f_c'^{\,0.5}}$

For HSC, Equation below was recommended by Carrasquillo et al. for the estimation of modulus of rupture of normal-density concrete from compressive strength

$$f_r\ (\text{MPa}) = 0.94\, f_c'^{\,0.5} \quad \text{for } 21\ \text{MPa} < fc' < 83\ \text{MPa}$$

Strength gain with age

HSC shows a higher rate of strength gain at early ages compared with lower-strength concrete, but at later ages, the difference is not significant (Fig. 4)

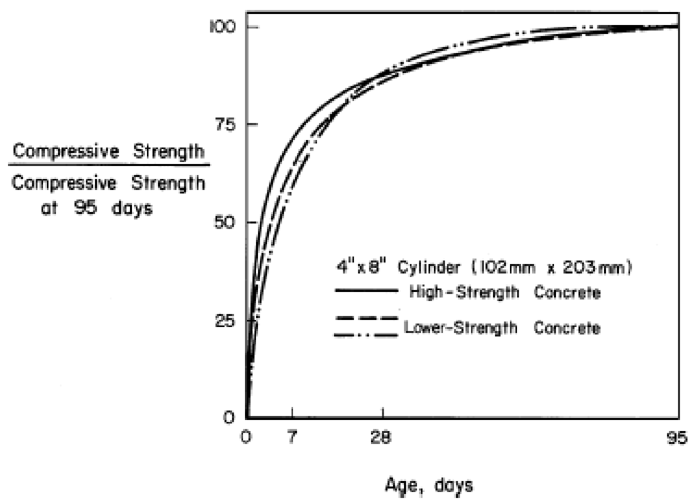

Fig. 4: Normalized strength gain with age.

Typical ratios of 7- to 28-day strengths of 0.8 to 0.9 for HSC and 0.7 to 0.75 for lower-strength concrete, whereas typical ratios of 7- to 95-day strength of 0.60 for low-strength concrete, 0.65 for medium-strength concrete, and 0.73 for HSC are reported.

Higher rate of strength development of HSC at early ages is caused by: 1) an increase in the internal curing temperature in the concrete cylinders due to a higher heat of hydration; and 2) shorter distance between hydrated particles in HSC due to a low w/cm.

Drying shrinkage

It is suggested that the drying shrinkage of HSC is similar to the shrinkage of normal-strength concretes, with values of 200×10^{-6} to 400×10^{-6} being reported.

Creep

It is reported that the total strain observed under sustained load in HSC was the same as that of lower-strength concrete when expressed as a ratio of the short-term strain.

STRUCTURAL DESIGN CONSIDERATIONS

HSC have some characteristics and engineering properties that are different from those of lower-strength concretes. These distinctions are increasingly important as strengths increase and should be recognized by design engineers in predicting the performance and safety of structures.

This chapter focuses on the design of structural members with design compressive strengths more than 8000 psi (56 MPa).

The use of HSCs permits efficient structural designs, allowing members to span longer distances, be smaller in cross section, and carry larger loads. These designs are likely to be controlled by serviceability and other practical design considerations instead of strength. As a result, special considerations may be required in the design of HSC structural members.

Concentrically loaded columns

Because the strength of columns is generally controlled by the compressive strength of concrete, there are significant advantages to using HSC in columns, especially those that carry axial loads alone.

Axial strength

Present design practice, in calculating the nominal strength of an axially loaded member, is to assume a direct addition law summing the strengths of the concrete and steel. The usual assumption is made that steel and concrete strains are identical at any load stage.

strength of the column is predicted by

$P = 0.85\, f'_c\,(A_g - A_{st}) + f_y A_{st}$

where f'_c is the specified compressive strength of the concrete; f_y is the yield strength of steel; A_g is the gross area of section; and A_{st} = total area of longitudinal steel

A similar analysis holds for HSC columns,

Beams and one-way slabs

The material properties of HSC can have a significant effect on certain aspects of structural performance for HSC beams.

Flexural strength

Fig. 5a shows the generally parabolic shape of the compressive stress distribution in a beam made of lower-strength concrete. For HSC, the stress-strain curve is more linear than parabolic, resulting in the compressive stress distribution shown in Fig. 5b.

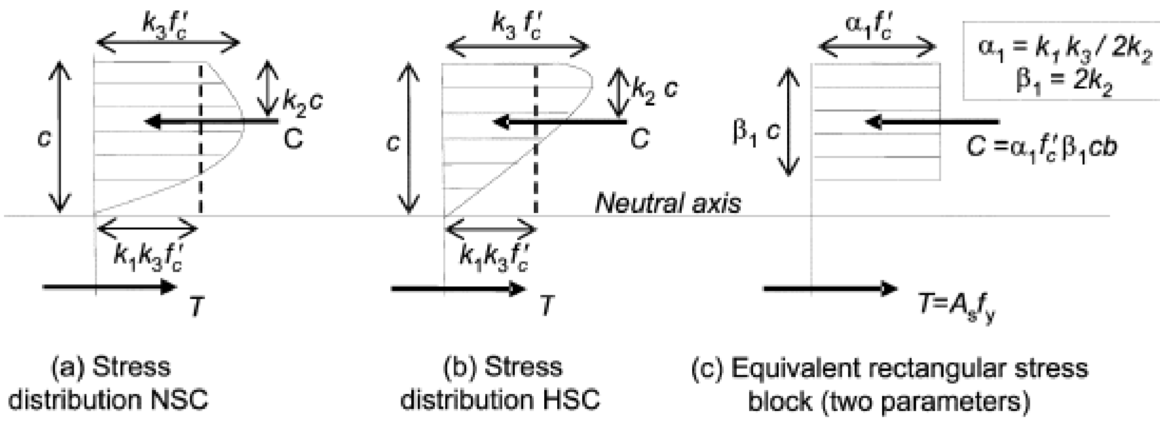

Fig. 5: Compressive stress distribution

For beams with typical levels of reinforcement, doubling the compressive strength of concrete used in the beam will generally only result in an increase of flexural strength of about 10%. This makes the use of HSC somewhat inefficient for under-reinforced (tension-controlled) beams.

For over-reinforced (compression-controlled) sections, which are not permitted by ACI 318 for flexural members, the flexural strength will be much more impacted by the compressive strength of concrete. The flexural strength of over-reinforced concrete sections can be enhanced greatly using HSC.

It is suggested that the existing ACI 318 equivalent rectangular stress block (Fig. 5c) is acceptable for use in the design of under-reinforced HSC beams.

ACI 318-19 specifies values of β_1 equal to 0.85 for concretes with a compressive strength of (28 MPa) or lower and 0.65 for concretes with a compressive strength of (56 MPa) or higher, with a linear relationship between β_1 and f'_c for intermediate values.

Limiting compressive strain and section ductility

Whereas HSC reaches its peak stress at a compressive strain slightly higher than that for lower-strength concrete, the ultimate strain is lower for HSC. The constant value of strain at extreme concrete compression fiber of **0.003** prescribed by ACI 318 is seen to satisfactorily represent the experimental results for HSC as well as lower-strength concrete.

Section ductility is important because it allows for plastic hinging to develop in beams. The formation of plastic hinges allows for adequate deformability to warn of impending failure and allows for the redistribution of moments in structurally indeterminate systems.

Ductility can be defined as a ratio of the deflection (or cross-section curvature) at failure to the deflection (or curvature) at the load producing yield of the reinforcement.

For under-reinforced HSC beams with quantities of reinforcement close to the balanced steel ratio, however, little ductility or plastic rotation capacity can be expected. For over-reinforced beams, little to no ductility should be expected, regardless of the concrete compressive strength.

ACI 318 sets a lower limit on the amount of tensile reinforcement to guard against sudden failure of very lightly reinforced beams upon concrete cracking, when the tension formerly carried by the concrete is transferred to the steel reinforcement. The ACI 318 expression for minimum steel ratio (ρ_{min}) is derived on the basis that the resisting moment of the cracked section should be at least as great as the moment that caused cracking, based on the modulus of rupture. Because the latter is known to be greater for HSC than for lower-strength concrete, ρ_{min} depends on f'_c:

$$\rho_{min} = 0.25 \sqrt{f'_c} / f_y \geq 1.4 / f_y$$

For $f'_c / f_y = 56 / 420$, $\rho_{min} = 0.25 \sqrt{56} / 420 \geq 1.4 / 420$
$= \underline{0.0045} > 0.0033$

Why HPCs are More Durable than Usual Concretes?

Consider two cement pastes with different water/binder (w/b) ratios (0.65 and 0.25) in both the fresh and hardened states. When hardened, each of these cement pastes has a different microstructure.

In a cement paste with a w/b = 0.65, which should give a concrete with $f'_c \approx 25$ MPa, the cement particles are quite far from each other compared with their respective positions in the paste having a w/b = 0.25, which should give a concrete of $f'_c \approx 100$ MPa. In the latter case the hydrated product fills most of the intergranular space, leading to a very rapid strength increase.

This difference in the microstructure of HPC has two very important consequences for its compressive strength and permeability. The f'_c of HPC concrete increases in a very spectacular manner and can reach 100 MPa, instead of 25 MPa for the higher w/b ratio concrete. Moreover, the permeability of HPC is considerably lower than that of usual concrete. HPC is so impervious to water that it is almost impossible to measure its water permeability.