High strength / high performance concrete
A historical perspective
1960. At that time most designers were satisfied to design structures based on the 15 to 30 MPa concretes that were well understood, economical and safe.

High strength concrete first started to be used in Chicago area in the early 1960s.
Even though the strength of the first high strength concretes seem quite modest (≈50MPa) by current standards, it should be remembered that, at that time, most commercial cements were ground much coarser than at present, and the commercial water reducers were mostly lignosulphonate-based.
The compressive strength, over a period of about 10 years, slowly and progressively raised from 15-30MPa to 45-65MPa.

This was the state of development of high-performance concrete technology in the early 1970s when superplasticizers based on polycondensates of naphtalene sulphonate were first introduced to the concrete market (1977).
Fig. 3.4 High-strength concrete shapes, new US skylines (from *High-Strength Concrete*, EB114.01T, Portland Cement Association, 1994).
Fig. 3.3 Concrete in high-rise construction (from Concrete Reinforcing Steel Institute, *Bulletin No. 4, 1990*).
Some High-Rise Buildings around the World

Eiffel Tower Chrysler Building Empire State Building John Hancock Building Sears Tower Center World Trade Center Hong Kong and Shanghai Bank First Interstate Bank Bank of China Bank Tower Petronas Towers
321 m 319 m 381 m 344 m 417 and 415 m 443 m 179 m 219 m 369 m 451 m

Fig. 3.5 Some high-rise buildings around the world, after *Le Monde* (Edelmann, 1996).
It should be mentioned that the first applications of superplasticizers were as fluidifiers, rather than as water reducers.

By using large enough dosages of superplasticizers it was found possible to lower the W/C ratio of concrete down to 0.30 and still get an initial slump of 200mm.
It was possible to decrease W/C ratio to 0.30, then to 0.27, then to 0.25 and recently down to 0.23 to obtain a compressive strength of 130MPa.

It was only in the late 1970s that silica fume started to be used as a supplementary cementitious material in concrete.
The particular advantage of using silica fume as a very fine and reactive pozzolan for use in high performance concrete was rapidly recognized.

By using silica fume, it has been shown that it is possible to make workable concretes with compressive strengths in the 100 to 150MPa range.
High performance concrete is not a super material, without any weaknesses and drawbacks. But as we learn more through its growing use, research, successful experiments and failures, it will be more effectively utilized to the advantage of us all.

High performance concrete makes concrete a better performing material, allowing designers to use it efficiently in increasingly slender structures.
Concrete with high strength
The way in which concrete transfers its forces can be schematically be imagined as shown in the figure.

Fig. 2. Schematic distribution of forces in concrete subjected to compression
The forces in the concrete go from aggregate to aggregate. Furthermore, transversal forces are transmitted by the binder material between the aggregates.

The concrete can be loaded uniaxially in compression until the binder material fails. Therefore the cracks in concrete loaded in compression develop in the direction of the compressive forces.
It is well known that the transversal forces inside the concrete can be minimized by realizing a dense packing of the aggregates. A second well known method in order to increase the strength is decreasing the W/C ratio.

The silica fume, a waste product of the silicium production, has a significant effect of the force transmission in the concrete.
The silica fume grains are a factor 100 smaller than the cement grains. This fact can be described as a filler effect.

The small grains fit into the space between the cement grains and thus increase the density of the paste, furthermore they are reactive.
The amorphous silica fume converse the weaker CH crystals in the stronger C-SH gel. The binder becomes much stronger and the transition zone between aggregate and paste is improved. The Young's modulus of the binder in much higher than before, thus the concrete is more homogeneous.
An analysis of the cost for using high strength concrete leads to a surprising result. The price for high strength concrete is about twice as much as for a conventional concrete. But considerable advantages have been found in contrast to the price of the concrete itself.
Because of the smaller surface of the cross-section of the structural elements, less concrete is necessary. Due to the fast hardening of the concrete, the construction time can be shortened. Because of the dense material structure of the high strength concrete, the durability is improved and the maintenance costs are expected to be less.
Realizing this, it is more and more talking about high performance concrete instead of high strength concrete.
<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Standard test method</th>
<th>FHWA HPC performance characteristic grade</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td>1</td>
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<td>2</td>
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<tr>
<td></td>
<td></td>
<td>3</td>
</tr>
<tr>
<td>Freeze-thaw durability(^4)</td>
<td>AASHTO T 161, ASTM C 666, Proc. A</td>
<td>70%≤F/T&lt;80%</td>
</tr>
<tr>
<td>(F/T=relative dynamic modulus of</td>
<td></td>
<td>80%≤F/T&lt;90%</td>
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<tr>
<td>elasticity after 300 cycles)</td>
<td></td>
<td>90%≤F/T</td>
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<tr>
<td>Scaling resistance(^5)</td>
<td>ASTM C 672</td>
<td>3.0&gt;SR&gt;2.0</td>
</tr>
<tr>
<td>(SR=visual rating of the surface</td>
<td></td>
<td>2.0&gt;SR&gt;1.0</td>
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<tr>
<td>after 50 cycles)</td>
<td></td>
<td>1.0&gt;SR&gt;0.0</td>
</tr>
<tr>
<td>Abrasion resistance(^6)</td>
<td>ASTM C 944</td>
<td>2.0&gt;AR≥1.0</td>
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<tr>
<td>(AR=avg. depth of wear in mm)</td>
<td></td>
<td>1.0&gt;AR≥0.5</td>
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<tr>
<td></td>
<td></td>
<td>0.5&gt;AR</td>
</tr>
<tr>
<td>Chloride penetration(^7)</td>
<td>AASHTO T 277, ASTM C 1202</td>
<td>2500≥CP&gt;1500</td>
</tr>
<tr>
<td>(CP=coulombs)</td>
<td></td>
<td>1500≥CP&gt;500</td>
</tr>
<tr>
<td></td>
<td></td>
<td>500≥CP</td>
</tr>
<tr>
<td>Alkali-silica reactivity(^8)</td>
<td>ASTM C 441</td>
<td>0.20≥ASR&gt;0.15</td>
</tr>
<tr>
<td>(ASR=expansion at 56 d) (%)</td>
<td></td>
<td>0.15≥ASR&gt;0.10</td>
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<tr>
<td></td>
<td></td>
<td>0.10≥ASR</td>
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<tr>
<td>Sulfate Resistance (SR=expansion)</td>
<td>ASTM C 1012</td>
<td>SR≤0.10 at 6 months</td>
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<tr>
<td>(%)</td>
<td></td>
<td>SR≤0.10 at 12 months</td>
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<tr>
<td></td>
<td></td>
<td>SR≤0.10 at 18 months</td>
</tr>
<tr>
<td>Flowability(^9) (SL=slump, SF=slump flow)</td>
<td>AASHTO T 119, ASTM C 143, proposed slump flow test</td>
<td>SL&gt;190 mm (SL&gt;7-1/2 in), and SF&lt;500 mm (SF&lt;20 in)</td>
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<tr>
<td>Strength(^8) (f'(_c)=compressive strength)</td>
<td>AASHTO T 22, ASTM C 39</td>
<td>55≤f'(_c)&lt;69 MPa (8≤f'(_c)&lt;10 ksi)</td>
</tr>
<tr>
<td>Elasticity(^9) (E(_c)=modulus of elasticity)</td>
<td>ASTM C 469</td>
<td>34≤E(_c)&lt;41 GPa (5≤E(_c)&lt;6×10^{6} \text{ psi})</td>
</tr>
<tr>
<td>Shrinkage(^10) (S=microstrain)</td>
<td>AASHTO T 160, ASTM C 157</td>
<td>800&gt;S≥600</td>
</tr>
<tr>
<td>(C=microstrain/pressure unit)</td>
<td></td>
<td>600&gt;S≥400</td>
</tr>
<tr>
<td></td>
<td></td>
<td>400&gt;S</td>
</tr>
<tr>
<td>Creep(^11) (C=microstrain/pressure unit)</td>
<td>ASTM C 512</td>
<td>75≥C&gt;55/MPa (0.52≥C&gt;0.38/\text{psi})</td>
</tr>
</tbody>
</table>
Mix design and proportions
(according Federal Highway Administration's)
HPC helps to optimize the construction process, but only after optimizing the concrete mixture for specific desirable properties.
The mix designer may require higher compressive strength, or specific requirements on shrinkage, creep or modulus of elasticity.
Mix design and proportions according to Federal Highway Administration's Exposure conditions may dictate concrete that has specific levels of resistance to sulphate attack, abrasion resistance, resistance to alkali-silica reaction or frost damage.
Mix design and proportions
(according Federal Highway Administration’s)

Mixture proportioning - cement

Typically use only Tipe I or Tipe II cement.
Look for medium range fineness in the cement, if cement is too fine, the concrete will produce excess heat during hydration.
The cements produced under the same specification may not perform the same.
Mix design and proportions
(according Federal Highway Administration's)
Mixture proportioning - cement
The mix designer will need to trial batch concrete with a specific cement and then continue to use this cement for the duration of project.
Mix design and proportions
(according Federal Highway Administration's)

Mixture proportioning - cement

HPC requires an understanding of the interaction of Portland Cement with Fly Ash or Silica Fume.

Usually the combination of Fly Ashes that are low in calcium and cement delays the setting time and thus the short-term strength development.
Fig. 1—Compressive strength versus age—Cubic specimens

Fig. 2—Compressive strength versus percentage fly ash—Cubic specimens
Mix design and proportions
(according Federal Highway Administration's)

*Mixture proportioning - cement*

Silica Fume has an extremely small surface area with the ability to react with calcium hydroxide to form a very hard and impermeable concrete.

However one short coming of this material is its propensity for shrinkage cracking (bleeding retarded).
Mix design and proportions
(according Federal Highway Administration’s)

*Mixture proportioning - aggregates*

Smaller maximum size aggregates are typically needed to ensure a high mortar to aggregate bond.

A general guideline should suggest that as the compressive strength increases, use 25mm to 10mm maximum size aggregate.
Mix design and proportions
(according Federal Highway Administration's)

Mixture proportioning - aggregates
Crushed aggregates are better than smooth, because of the angular surfaces that are formed as a result of the crushing process.
The rough angular surfaces forms a strong bond at the aggregate/cement paste interface.
Mix design and proportions
(according Federal Highway Administration’s)

Mixture proportioning - aggregates
Fine aggregate is also a very important part of the mixture.

In general HPC can be produced by using natural or uncrushed rounded sands, with a fineness modulus between 2.6 and 3.1. When designing HPC, the ratio of the fine to coarse aggregate is reduced.
Mix design and proportions
(according Federal Highway Administration's)

Mixture proportioning - aggregates

The workability of concrete is assisted by the use of high range water reducing admixtures.
Mix design and proportions
(according Federal Highway Administration’s)

Mixture proportioning - admixtures

The purpose of high range water reducing admixtures (HRWR) is to give additional time to place concrete and to delay the setting time.

HRWR will normally reduce early strength at 24 hours, but after this initial delay, typical strength gain occurs.
Mix design and proportions
(according Federal Highway Administration's)

Mixture proportioning - admixtures
The dosage rate of HRWR will typically be higher than normal concrete mixtures.
If a newer generation of HRWR are utilized (admixture designed for flowing and SCC) the HRWR usage rates will be defined by trial batching the concrete and identifying the plastic properties of the mixtures.
Mix design and proportions
(according Federal Highway Administration’s)

Mixture proportioning - admixtures

If the concrete will be exposed to cycles of freezing and thawing, air entrainment admixture will be needed to allow the concrete to expand and contract.
Mix design and proportions
(according Federal Highway Administration’s)

Mixture proportioning - admixtures

A typical amount of air entrainment is approximately 3 to 4%. However air entrainment will reduce the strength of concrete (5-7%).
Early cracking

High performance concretes have the reputation of being prone to develop early cracking. HPC have a low water/cement or water/binder ratio and this makes a great difference from a volumetric stability point of view. Basically HPC have a w/c or w/b ratio lower than 0.4. The lower w/c or w/b the earlier and greater the development of autogenous shrinkage.
Time-development of autogenous and drying shrinkage in normal and high-performance concrete
Early cracking

Moreover, low w/c or w/b concretes after their placing do not bleed, therefore there are prone to develop plastic shrinkage if not appropriately cured.
How to minimize early cracking

From a practical point of view HPC having w/c or w/b ratio close to or slightly higher than 0.36 and cured in the presence of external source of water are rather robust towards early cracking.

In order to make HPC structures durable, plastic shrinkage and autogenous shrinkage must absolutely be eradicated during the first hours following its placing.
How to minimize early cracking

Fog spraying or the application of an evaporation retarder must be carried out to prevent a plastic shrinkage.

As the autogenous shrinkage is linked to cement hydration, water curing must start as soon as hydration starts.
Mechanical properties

Stress-strain curve in compression

The stress-strain curve for HSC is different than that for normal strength concrete. As the concrete strength increases, the concrete stress-strain curves exhibit increased initial stiffness and greater linearity. In HSC microcracks occur only at higher load levels than in concrete of normal strength, thus the stress-strain curves show a linear elastic shape over a large area.
Mechanical properties

Stress-strain curve in compression

The strain under maximum load grows with increasing concrete strength.

After the maximum load has been exceeded the stresses drop steeply.

The portion of the energy consumption in the post peak area is lower when compared to concrete of normal strength.
Variation of compressive stress-strain curves with increasing compressive strength
Stress-strain diagram for uniaxial compression

plasticity number:

\[ k = \frac{E_{ci}}{E_{c1}} \]
### Tangent modules \( E_{ci}, E_{c1}, \varepsilon_{c1} \) and \( \varepsilon_{c,lim} \) for various concrete grades

<table>
<thead>
<tr>
<th>Concrete grade</th>
<th>C12</th>
<th>C20</th>
<th>C30</th>
<th>C40</th>
<th>C50</th>
<th>C60</th>
<th>C70</th>
<th>C80</th>
<th>C90</th>
<th>C100</th>
<th>C110</th>
<th>C120</th>
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<tbody>
<tr>
<td>( E_{ci} ) [GPa]</td>
<td>25.8</td>
<td>28.9</td>
<td>32.0</td>
<td>34.6</td>
<td>36.8</td>
<td>38.8</td>
<td>40.7</td>
<td>42.3</td>
<td>43.9</td>
<td>45.3</td>
<td>46.7</td>
<td>48.0</td>
</tr>
<tr>
<td>( E_{c1} ) [GPa]</td>
<td>10.5</td>
<td>13.5</td>
<td>17.0</td>
<td>20.3</td>
<td>23.4</td>
<td>26.3</td>
<td>29.2</td>
<td>31.9</td>
<td>34.6</td>
<td>37.2</td>
<td>39.8</td>
<td>42.7</td>
</tr>
<tr>
<td>( \varepsilon_{c,lim} ) [%]</td>
<td>-3.5</td>
<td>-3.5</td>
<td>-3.5</td>
<td>-3.5</td>
<td>-3.4</td>
<td>-3.3</td>
<td>-3.2</td>
<td>-3.1</td>
<td>-3.0</td>
<td>-3.0</td>
<td>-3.0</td>
<td>-3.0</td>
</tr>
<tr>
<td>( k = E_{ci}/E_{c1} )</td>
<td>2.46</td>
<td>2.14</td>
<td>1.88</td>
<td>1.71</td>
<td>1.58</td>
<td>1.48</td>
<td>1.39</td>
<td>1.33</td>
<td>1.27</td>
<td>1.22</td>
<td>1.17</td>
<td>1.12</td>
</tr>
</tbody>
</table>
Stress-strain diagrams for different concrete strengths
Mechanical properties

Tensile strength

A significant effect of the increase in compressive strength is the decreasing ratio of the direct tensile strength versus the result of the standard splitting test.

So the conventional determination of tensile strength may overestimate the real tensile properties or cracking limit of concrete.
Mechanical properties

Long term evaluation of tensile strength

It was noted a systematic loss in tensile strength statistically related to compressive strength, reaching about -10% for 80MPa compressive strength and -20% for 120MPa.

It seems reasonable to relate this loss to autogenous shrinkage, which increases with compressive strength and leads to internal cracking due to the stiffness of aggregates.
Durability performance of HPC

A large amount of experimental data indicate that as the compressive strength increases from 25MPa to 100MPa the porosity accessible to water is reduced up to about 50%, the chloride diffusion factor is significantly reduced, carbonated dept after 28 days, significant for normal strength concrete, is not detectable also after 90 days for HPC (>50MPa).
Pore size distribution of UHPC, HPC and Normal Strength Concrete
Chloride diffusion values of UHPC, HPC and Normal Strength Concrete
Scaling of UHPC under freeze-salt attack compared with air entrained concretes
Self compacting concrete
A large disadvantage of conventional concrete is that it has to be mechanically compacted. This compacting is an unpleasant job. Also in this case, a small change in the mix composition, can lead to a significant improvement. The idea is rather simple. One adds a little more binder than is necessary for a good performance of the concrete in hardened state.
By this small amount of additional cement paste, small layers of **lubrificant** develop around each grain. The grains start to float. The inner friction during the flow of the concrete is further reduced due to the presence of **fly ash** or **limestone powder** as an extra component.
“Floating grains” in self-compacting concrete
These materials have a grain diameter which roughly fits between sands and cement. A three phase particle skeleton with high density is formed. The grains move and roll easily over and along each other, which makes the material very flowable. Due to the dense grain packing, the strength is automatically relatively high.
Schematic concept of the three component grain skeleton
On the other hand the mixtures are rather sensitive for small changes in the properties of the ingredients. It is therefore not surprising that the application of self compacting concrete mainly develops in the prefab industry. The circumstances in a factory are relatively constant and actions are repeated.
It is true that the self compacted concrete is a little more expensive due to the extra costs for superplasticizer and filling agents. But considerable savings compensate for these costs due to decrease of energy consumption, decrease in maintenance costs, longer life time of the moulds.