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Potential applications for high-strength concrete in cast in-situ structures

Master's thesis submitted in partial fulfilment of the requirements for the degree of Master of Science in Technology.

Espoo 18.02.2019
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Abstract of master's thesis

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Title of thesis Potential applications for high-strength concrete in cast in-situ structures

Master programme Building technology **Code** CIV

Thesis supervisor Professor of Practise Jouni Punkki

Thesis advisor(s) DI Elina Paukku

Date 18.02.2019 **Number of pages** 75 + 8 **Language** English

High-strength concrete (HSC) is a concrete class with higher compressive strengths than that of commonly used normal-strength concrete (NSC). Although the limits defining HSC are continually changing as concrete in general increases in strength, but today HSC can be defined as concrete with cylindrical compressive strengths in the range of 50-100 MPa. Nevertheless, HSC does not only possess superior compressive strengths compared to NSC, but also the modulus of elasticity and general durability are improved when HSC is used appropriately. The advantages of using HSC in demanding construction projects has been recognized worldwide ever since it was introduced in the 1960s, enabling the construction of even higher skyscrapers in the US. However, since then, HSC has continued to primarily be used for extraordinary structures and buildings only.

Therefore, this thesis aims to explain why the application areas of HSC has remained relatively narrow and whether using HSC for a wider range of structures and purposes can be beneficial. The unwillingness of using HSC further can in many cases be attributed to justified or unjustified prejudices connected to the use of this material. Although the general durability of structures is improved by using HSC, the fire resistance of plain HSC can undoubtedly be poorer than that of NSC. Also, the freeze-thaw resistance of concrete structures is indeed improved when using HSC, but to what extent is not fully established. Furthermore, beliefs that HSC require more care throughout the construction process, accompanied by that higher initial prices of this material will lead to higher total costs, are also aspects that need to be addressed to encourage further use of HSC.

To demonstrate that the use of HSC instead of NSC does not result in higher overall costs a comparison was conducted evaluating how the use of different concrete strength classes affects the dimensions and final costs of various column cases. These evaluated concrete classes were also produced in laboratory conditions to study the actual strength and temperature developments. The production and testing of these concretes gave some indications on how easily obtainable the design strengths are and what other challenges should be considered when producing HSC. Although the experimental production of the concrete classes showed that reaching the desired properties of both fresh and hardened concrete can be challenging, the column comparison indicates that significant economic benefits are attainable by using higher strength concrete classes.

Keywords high-strength concrete, high-performance concrete, reinforced concrete columns, economic design

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Datum 18.02.2019 **Sidantal** 75 + 8	**Språk** Engelska

Höghållfasthetsbetong är en typ av betong som besitter en högre tryckhållfasthet än vad som är förknippat med vanligen använd betong. Även om gränserna för vad som kan betraktas som höghållfasthetsbetong ständigt ändras i takt med att betongens hållfasthet överlag ökar, kan man idag definiera höghållfasthetsbetong som betong med en cylindrisk tryckhållfasthet mellan 50–100 MPa. Det är dock inte endast hög tryckhållfasthet som kännetecknar höghållfasthetsbetong, även högre elasticitetsmodul och generellt bättre hållbarhet är förknippat med god användning av höghållfasthetsbetong. Fördelarna med användningen av höghållfasthetsbetong för krävande byggprojekt har erkänts världen över ända sedan materialet introducerades och möjliggjorde byggandet av högre skyskrapor på 1960-talet i USA. Sedan dess har likväl höghållfasthetsbetong huvudsakligen tillämpats endast vid uppförandet av extraordinära byggnader.

Denna avhandling ämnar därför förklara varför tillämpningsområdena för höghållfasthetsbetong har förblivit relativt få, och huruvida en bredare tillämpning av detta material kan vara fördelaktigt. Oviljan att använda höghållfasthetsbetongen ytterligare bottnar ofta i antingen grundade eller ogrundade fördomar angående materialet. Även om den generella hållbarheten ökar vid användning av höghållfasthetsbetong, minskar onekligen brandbeständigheten jämfört med konventionell betong. Frostbeständigheten hos höghållfasthetsbetong är å andra sidan högre än hos vanlig betong, men till vilken grad är inte fullständigt fastställt. Dessutom måste aspekter som att höghållfasthetsbetong är mer krävande att gjuta och att användningen av detta material leder till högre totalkostnader adresseras för att främja en bredare användning av detta material.

För att demonstrera att användningen av höghållfasthetsbetong, jämfört med konventionell betong, inte ger högre totalkostnader genomfördes en utvärdering angående hur betongens hållfasthet påverkar kostnader och dimensionering av pelare. De jämförda betongklasserna producerades även experimentellt för att studera den verkliga hållfasthetsutvecklingen samt temperaturproduktionen. Framställningen och proverna av betongerna gav indikationer om hur lättuppnåeliga de eftersträvade materialegenskaperna är, samt vilka utmaningar produktionen av höghållfasthetsbetong medför. Även om den experimentella produktionen av de olika betongklasserna visade att för att uppnå målsättningarna bör produktionsprocessen optimeras för den tillgängliga utrustningen, visar pelarjämförelsen att användningen av höghållfasthetsbetong kan medföra märkbara ekonomiska fördelar.

Nyckelord höghållfasthetsbetong, högpresterande betong, armerade betongpelare, ekonomisk design

Preface

This Master's thesis has been conducted as part of the degree Master of Science in Technology at the Aalto University, School of Engineering.

The topic of the thesis was initially intended to investigate whether, and how, high-strength concrete could be utilized in broader range of application areas. However, it was quickly realized that this thesis should focus mainly on cast in-situ structures, and especially areas where high-strength concrete is commonly applied and most beneficial. Thus, the emphasis of this thesis primarily fell on high-strength concrete in load bearing elements, such as columns, and how the use of this material can be optimized.

I would like to thank my supervisor Jouni Punkki, advisor Elina Paukku as well as unofficial advisor Mika Tulimaa for guiding and supporting me throughout the whole process. Additional thanks go to the helpful personnel of the civil engineering departments laboratory, where the experimental part of this work was conducted.

For the financial support of this thesis, as well as providing knowledgeable and supportive people to ask advice from, I would like to thank Sweco Rakennetekniikka Oy.

Espoo, February 2019

Hannes Rolander

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Symbols

A_c	gross area of concrete section
A_s	area of longitudinal reinforcement
$A_{s,min}$	minimum area of longitudinal reinforcement
$A_{s,act}$	actual area of longitudinal reinforcement
E_c	modulus of elasticity for concrete
M_{Ed}	design bending moment
M_{Rd}	design moment resistance
N_{Ed}	design axial force
N_{Rd}	design axial resistance
V_c	volume of concrete
b	width of column cross-section
e_0	minimum eccentricity
f'_c	specified compressive strength of concrete
f'_{cf}	characteristic flexural tensile strength of concrete
f'_{ct}	characteristic principal tensile strength of concrete
f_c	cylinder compressive strength of concrete
f_{ck}	concrete 28-day characteristic compressive cylinder strength
f_{cu}	cube compressive strength of concrete
f_{yk}	characteristic yield strength of reinforcement
f_{yd}	design yield strength of reinforcement
h	depth of column cross-section
ø	diameter of reinforcement bar
α_{CC}	coefficient for unfavourable effects on concrete strength
γ_C	partial factor for concrete
γ_S	partial factor for reinforcing steel
Δc_{dev}	accepted deviation
ε_{cu}	ultimate compressive strain in concrete
ρ_c	density of concrete

Abbreviations

AAR	alkali-aggregate reaction
ACI	American Concrete Institution
AS	Australian Standards
ASCE	American Society of Civil Engineers
ASR	alkali-silica reaction
BS	British Standards
BY	Suomen Betoniyhdistys
CSA	Canadian Standards Association
CTBUH	Council on Tall Buildings and Urban Habitats
EC	Eurocodes
EN	European Standards
fib	The International Federation for Structural Concrete
FA	fly-ash
FRC	fibre reinforced concrete
FRHPC	fibre-reinforced high-performance concrete
GGBFS	ground granulated blast-furnace slag
HPC	high-performance concrete
HSC	high-strength concrete
HRWRA	high-range water-reducing admixture
IC	internal curing
ITZ	interfacial transition zone
NSC	normal-strength concrete
RC	reinforced concrete
RILEM	International Union of Laboratories and Experts in Construction
SCC	self-consolidating concrete
SCM	supplementary cementitious materials
SF	silica fume
SFS	Suomen Standardisoimisliitto ry
SP	superplasticizer
UHPC	ultra-high-performance concrete
w/c	water-cement ratio
w/cm	water-cementitious materials

1 Introduction

As the construction industry constantly develops, the building materials naturally follows this development. As the second most widely used building material in the world, concrete have naturally evolved as well. An indication of this evolution are the various subcategories that have been developed to increase the efficient use of this material. One such subcategory is high-strength concrete (HSC), which has been of increasing interest all over the world since its introduction in the mid-1960s (Aïtcin, 1998).

It is well known that concrete possesses good compressive strength, therefore, high-strength concrete focuses on improving this attribute even further. According to the American concrete institution (ACI), high-strength concrete is defined as concrete with design compressive strength of 55 MPa or higher (ACI CT-13, 2013). This compressive strength value refers to the commonly recognized strength of a cylindrical specimen with the dimensions of 150 x 300 mm, equivalent to the first value in the European concrete strength classification. Additionally, concrete compressive strength can also be derived from cubical specimens with the dimensions 150 x 150 mm, the second value in the European classification, or 100 x 100 mm.

While high-strength concrete is not exactly specified in the Eurocodes (EN 1992-1-1, 2004), it is fair to assume that the lower limit of compressive strength class for HSC is C50/60, since the formulas and measurements change for concrete classes above this limit. When the compressive strength of concrete reaches the regions of 120 MPa and above, the properties of the material has been modified to the extent that it is classified as ultra-high-performance concrete (UHPC).

High-strength concrete is commonly used for columns or other loadbearing members that are exposed to high compression forces. Therefore, typical structures include high-rise buildings where the use of HSC enables smaller structural members, thus increasing the valuable floor area, which is essential especially on the lower floors. The smaller dimensions of structural members, made possible by using HSC, is also of great importance in reducing the overall weight of the building. Other favourable application areas for HSC include harsh environments, such as infrastructure and marine structures, as well as structures requiring a general high durability. For these kind of application areas, the low permeability of HSC is often the attribute of importance.

Although the strength and permeability of concrete is closely connected, the desire for good durability, or other improved property, can lead to the use of the broader term high performance concrete (HPC), instead of focusing on the high strength of the concrete. However, due to the strong correlation between high strength and low permeability, which both are mainly due to the low water-cementitious materials ratio (w/cm ratio), HPC and HSC are often used synonymously. Henceforth, the term high-strength concrete will be used in this thesis, but it is important to acknowledge the other improved key properties of concrete that HSC possess in addition to a high compressive strength. These properties should also be considered when justifying the use of HSC instead of normal-strength concrete (NSC).

Although the use of high-strength concrete has many advantages, these kinds of concrete mixes also require extra precautions and are often considered more demanding to cast and cure than normal-strength concretes. Therefore, the use of HSC has mainly been limited to application areas in which its use is essential, and where normal concrete would not suffice.

Compared to NSC, the additional requirements for HSC have been a subject of much research for the recent decades. It has been established that HSC concrete requires careful attention to the curing process as well as strict quality control (Aïtcin, 1998; Paulik, 2013). Research has also shown that air-entraining is not necessarily needed in order to achieve frost resistance in HSC (Hale et al., 2009). Even so, air-entraining is required for making frost resistant HSC according to many guidelines. Furthermore, research concerning the economical aspect of using HSC has also been conducted, proving particularly the financial benefits of using HSC in columns (Yousry and Shikh, 2008). Alves et al (2004) also demonstrated that existing proportioning methods for HSC might not always be the most efficient. Thus, material consumption may vary notably depending on the guidelines used for producing concrete of the same class with equivalent properties. This being important from both a sustainability as well as financial point of view.

Although the use of high-strength concrete is more or less required in demanding construction projects, such as high-rise buildings, its potential might not be fully utilized in more ordinary structures. The use of higher strength concrete could both lead to direct savings in material and space as well as improve the durability and sustainability aspects of the structure. However, due to the lack of national guidelines and experience of using high-strength concrete, designers may be reluctant to use high-strength concrete in cases where it is not absolutely required. Additionally, to encourage a broader use of HSC, it would be important to further develop available standardized ready-mix concretes for Finnish environments and purposes.

The aim of this thesis is to investigate at what extent high-strength concrete can or should be utilized in different structural applications. Structures common for the Finnish building industry, or areas with similar conditions, will primarily be considered. Furthermore, the thesis also aims to specify how the use of high-strength concrete differs from normal-strength concrete, in order to determine what additional considerations and arrangements are necessary for reaching the full potential of this high-strength material. However, the thesis will principally focus on cast in-situ structures that can be realized with HSC suitable for Finnish environments utilizing raw materials that are available in the region.

In addition to the literature review, practical testing will be conducted to reach an understanding on which high-strength concrete types are realistically achievable using methods and materials familiar to Finnish concrete production. The experimental work includes testing of both fresh and hardened concrete properties of different HSC mixes that could be used to improve the economic as well as the quality aspect of mainly reinforced concrete (RC) columns.

This thesis will focus primarily on high-strength concrete with compressive strengths ranging from around 50 MPa to 100 MPa. Higher strength classes can be considered ultra high-strength concrete and will not be included in the thesis since they differ considerably from HSC and are difficult to produce simply by lowering the w/cm ratio. The range 50 – 100 MPa is also a suitable range for the thesis because the concrete classes within can be obtained in Finland but are rarely extensively used in construction projects. As higher buildings are emerging also in Finland, it is worth asking the question if this range of concrete strength classes should be utilized more widely.

The remainder of this thesis is divided into the following chapters. Chapter 2 introduces high-strength concrete along with its uses and guidelines. Chapter 3 reviews the various design considerations connected to high-strength concrete and approaches to optimize the use of this material. Chapter 4 comprises of a column comparison case study. In Chapter 5, the performed experimental work of this thesis is described and the results of the conducted experiments are presented and analysed. Finally, Chapter 6 discusses concluding remarks.

2 Background

2.1 History and development of high-strength concrete

Concrete is an ancient material, its earliest uses dating back to as early as 7000 BC. This concrete-like material, found in Israel, made up the floor of a hut and is thought to be made of burned limestone mixed with water and stone particles. The key ingredient of this material was the burned limestone, which possesses cementitious properties binding together the solid materials (Singh, 2017). Thereafter, lime and various other materials, such as mud, straw and rice, was utilized in different forms and compositions to produce concrete-like structures throughout the ancient civilizations. The primitive mortar was constantly improved by adding locally available materials to enhance the cementitious properties, e.g. the Romans utilizing these lime-based cementitious materials while adding a pozzolanic volcanic ash. However, it was not until the introduction of Portland cement in 1824 by Joseph Aspdin, that the modern concrete was truly established.

With the introduction of Portland cement, research was initiated on understanding the reactions responsible for the desired properties of the cement and how to produce it in an optimal way (Singh, 2017). This naturally led to the optimization of the composition and microstructure of concrete to maximize the favourable properties of this evolving material. Although concrete constantly developed, and its behaviour was fairly well understood, the compressive strength of concrete remained low for a long time. However, the connection between the compressive strength of concrete and the so-called water-to-cement ratio, w/c ratio, of concrete was well known, i.e. higher strength concrete required a lower w/c ratio (Aïtcin, 1998). To produce higher strength concretes than was normal at that time therefore called for a way to effectively reduce the use of water but keep the concrete mix workable. Although lignosulfate-based water-reducing admixtures had been introduced in the 1930s, these additives were not effective enough to lower the w/c ratio sufficiently to achieve high-strength concretes without severely affecting workability. Therefore, it was not until to the invention of high-range water-reducing admixtures (HRWRA), or superplasticisers, in the 1960s that concrete could reach w/c ratios as low as required for producing high-strength concrete while still being sufficiently workable.

These new superplasticisers were introduced almost simultaneously, in the early 1960s, in both Japan and Germany, and they were based on beta-naphtalene and melamine, respectively (Kawai, 2002). However, high range water reducers based on similar compounds (polycondensates of naphthalene sulfonate) had been patented as early as 1938 in the US (Aïtcin, 1998). But since the much more economical lignosulfate-based regular water reducers were regarded as satisfactory for producing the required concrete of the time, the utilization of superplasticisers did not seem necessary until later in the 1960s when the construction of high-rise buildings truly accelerated.

The use of superplasticisers resulted in various problems naturally following the introduction of a new technology, most notably the so-called slump loss (Kawai, 2002). The remarkable slump loss connected with the use of the early superplasticizers was a big concern for keeping the concrete workable and maintaining the castability even after longer transportation.

In Japan, this concern was solved by controlling the slump with a reactive polymeric dispersant. Although this concern has since been a topic of research and it is today known how problems connected to slump loss in the production of high-strength concrete can be avoided, the suitability of certain superplasticizers in combination with the other concrete raw materials must be investigated prior to production.

Another important factor in the development of high-strength concrete was the introduction of silica fume, also known as micro silica or condensed silica fume (Aïtcin, 1998). Silica fume is a by-product of the silicon and ferrosilicon production, and was not commercially available at a large extent until stricter environmental regulations were introduced in the 1970s which made the filtration and collection of these, at the time problematic, fine dust particles mandatory (Fidjestol and Dastol, 2008). Once the initial problems of collecting and storing these very fine particles had been overcome, the possibilities of using this material in the concrete industry was quickly further investigated. In fact, the advantages of using microsilica for concrete production had been investigated already in the 1940s and had since been the topic of sporadic research and used successfully in occasional projects. However, it was not until the abundance of silica fume became a reality in later years it received its well-deserved recognition as an extraordinary supplementary cementitious material (SCM).

The reasons for these excellent properties of silica fume, mainly improving the strength and durability of concrete, are due to the so called micro-filler effect and the pozzolanic reactivity (Kawai, 2002). Therefore, the use of silica fume is, if not required, certainly recommended when producing HSC, especially when the concrete is desired to reach compressive strengths of 80 MPa and above. However, from being a relatively cheap replacement for cement when introduced in the 1970s and 80s, the silica fume is currently mostly used for special concretes and when extraordinary strengths are required. Yet, supplementary cementitious materials, such as fly ash and ground granulated blast furnace slag (GGBFS), are increasingly exploited in the concrete industry. Both fly ash and GGBFS are by-products of industry making the use of these materials both economically and environmentally profitable. These materials also possess cementitious qualities, although less reactive than ordinary Portland cement and considerably less reactive than silica fume.

The effectivity of each supplementary cementitious material can be easily perceived by the so-called k-value, which is used in the Eurocodes. This k-value describes the performance of the SCMs compared to ordinary Portland cement. The k-values for silica fume, fly ash and GGBFS are in Finland generally considered to be 2.0, 0.4 and 0.8/1.0, respectively. The k-value for GGBFS may be taken as 0.8 or 1.0 depending on the relevant exposure class (BY 65, 2016). However, these values may vary for different national standards and circumstances. Also, the SCMs may be added to the cement content simply without factors, in these cases the cement plus SCMs are often considered as the total binder of the concrete. Therefore, it is important to recognize what a presented w/c, w/cm or w/b ratio takes into consideration. In this thesis a w/cm ratio that considers the cement content plus present SCMs multiplied by the relevant k-value will be preferred. Furthermore, the water to only cement and water to binder (cement plus SCMs without correction factor) ratio will be annotated as w/c and w/b ratio, respectively, henceforth in this thesis. The fact that FA and GGBFS are less reactive than cement also means less heat of hydration generated. This can be an important aspect, and an incentive for using SCM. Especially when concretes with a high cement content are produced, such as HSC, and the temperature of the concrete must be kept at a reasonable level during curing.

The first extensive use of high-strength concrete was seen mainly in the construction of high-rise buildings and bridges. The use of HSC took off in the US, and more specific in the Chicago area, in 1965 with the construction of the Lake Point Tower in Chicago which is considered the first building to incorporate HSC. The HSC used for some of the columns in this building had a compressive strength of 52 MPa, which in modern context would perhaps not be considered remarkable, but at the time it was very innovative. This ground-breaking and successful use of higher strength concrete in high-rise buildings was quickly utilized and developed further as many similar buildings with rising concrete strengths were constructed in that area in the following years (Gjørv, 2008). Basic details of some of the early buildings where HSC was used, mainly for load bearing columns, are presented in Table 2.1.

Table 2.1. List of early high-rise buildings incorporating HSC (Gjørv, 2008).

Building	Location	Year+	Total stories	Max design strength (MPa)
Lake Point Tower	Chicago	1965	70	52
Midcontinental Plaza	Chicago	1972	50	62
Frontier Towers	Chicago	1973	55	62
Water Tower Plaza	Chicago	1975	79	62
Royal Bank Plaza	Toronto	1975	43	61
River Plaza	Chicago	1976	56	62++
Helmsley Palace Hotel	New York	1978	53	55
Richmond – Adelaide Toronto	Centre	1978	33	61
Larimer Place Condominiums	Denver	1980	31	55
Texas Commerce Tower	Houston	1981	75	52
City Center Project	Minneapolis	1981	52	55
Trump Tower	New York	–	68	55
499 Park Avenue	New York	–	27	59
Petrocanada Building	Calgory	1982	34	50
S.E. Financial Center	Miami	1982	53	48
Chicago Mercantile Exchange	Chicago	1982	40	62+++
1130 A.Michigan Ave.	Chicago	–	–	52
Pacific Park Plaza	Emeryville, CA	1983	30	45
Collins Place	Melbourne	–	44	55
Columbia Center	Seattle	1983	76	66
Interfirst Plaza	Dallas	1983	72	69
900 N. Mich. Annex	Chicago	1986	15	97
Grande Arche de la Défence	Paris	1988	–	65
South Wacker Tower	Chicago	1989	79	83
Two Union Square	Seattle	1989	58	115
Pacific First Center	Seattle	1989	44	115
Gateway Tower	Seattle	1989	62	94

+ Year in which high strength concrete was cast.
++ Two experimental columns of 76 MPa strength were included.
+++ Two experimental columns of 97 MPa strength were included.

Simultaneously high-strength concrete was also being introduced in bridge construction, firstly in the US and Japan but eventually also in Europe. In Japan, HSC was at the beginning almost exclusively used for bridge structures, and mainly in the form of precast elements. This was the result of the initial slump-loss problems which made cast-in-situ applications difficult and thus limiting the use of HSC to factory produced products. However, the advantages of using HSC were clearly observed, in addition to the obvious increased strength and rigidity also the reduced dead weight of structural members was of great importance in bridge construction. Thus, as the problems concerning slump-loss were eventually overcome, the application areas for HSC in Japan increased further (Nagataki, 1997; Kawai, 2002).

In Europe, the first use of high-strength concrete was in the early 1970s for off-shore structures in Norway and the North Sea. As concrete had not until then been utilized in the off-shore construction, it had to be proven that concrete could be a durable enough material for these challenging environments. The need for a durable concrete that would be acceptable for this purpose therefore drove the development of HSC forward in Norway. However, in this context it is perhaps more correct to refer to the concrete as high-performance concrete, rather than high-strength concrete, since this concrete mainly had to perform very well in a durability aspect. Ever since the first concrete off-shore structure in 1973, the concrete used for similar structures, as well as a large portion of the infrastructure in Norway, has generally been HPC, making Norway a true authority and pioneering country regarding high-strength concrete (Helland, 1989; Gjørv 2008).

2.2 Comparing and defining high-strength concrete, high-performance concrete and regular concrete

Naturally, the definition of high-strength concrete (HSC) has changed during time accordingly with the development of concrete, as the strength of conventionally used concrete has risen, so has the limit defining HSC. Although there are some slight geographically differences regarding the definition of HSC even today, most standards having their own definition, it can be assumed that concrete possessing a cylindrical compressive strength of around 60 MPa and above is regarded as HSC.

This makes for a suitable limit in several aspects. Firstly, concrete without any special admixtures can with proper attention to casting and curing, achieve such compressive strength classes quite easily. Secondly, concrete with a cylindrical compressive strength below 60 MPa is usually adequate for ordinary concrete applications. In addition, concrete with a compressive strength over this start behaving mechanically differently, i.e. showing a more brittle nature, as well as changes in shrinkage and creep behaviour can be observed. However, even though the compressive strengths of normal-strength concrete (NSC) might be adequate, the use of HSC can lead to benefits in other aspects, such as more durable and economical structures. This will be investigated closer later in the thesis.

Since HSC possesses many other advantageous properties than only high strength, the use of the term high performance concrete (HPC) has become more common. The denser microstructure that gives concrete high strength also gives the concrete a lower permeability, thus making it more durable. In some cases, the improved durability might be the required property, although normal-strength concrete would suffice strength-wise, this making the use of the term HPC more fitting than focusing only on the improved strength. However, as predicted by Aïtcin (1998), concretes can have a high performance in aspects that does not necessarily affect the strength of the concrete. Such concretes can for example be self-healing concrete and self-consolidating concrete, both which can definitely be considered high performing concretes without necessarily having a high compressive strength. To be able to understand the difference between NSC and HSC, or HPC, and how to benefit the most from the higher performing concretes the focus should be put on the singular most determining difference, the microstructure.

Generally, no extraordinary materials are required to produce high-strength concrete. The same raw materials as for producing NSC, but in optimized proportions, are sufficient also for producing HSC. The key proportioning factor being the earlier mentioned w/c ratio, i.e. the mass ratio between water and cement. Even more useful is the w/cm-ratio, which also takes into consideration the possible supplementary cementitious materials present in the concrete, in addition to the pure cement. By simply reducing the w/cm-ratio concrete can in theory achieve compressive strengths up to around 100 MPa, but not higher (Breitenbücher, 1998). However, making concretes with this low w/cm ratios and without additives, means that the workability is very much compromised leading to a practically unusable fresh concrete. Therefore, the use of plasticizers, or even superplasticizers, is necessary in the production of HSC. Not for improving the mechanical properties of the hardened concrete, but simply making it possible to obtain as low a w/cm-ratio needed and still maintaining a sufficiently workable fresh concrete mass.

The low w/cm ratios, generally in the range of 0,2-0,4 for HSC, is such a decisive indicator of the performance of concrete that Aïtcin (1998) proposed that this should be the definition of HPC, and consequently also for HSC, rather than the compressive strength. However, it can be challenging to calculate the exact w/cm-ratio that accurately predicts the behaviour of the concrete. Partly because varying moisture content and absorption of aggregates used can be difficult to accurately monitor, but also determining universally correct reactivity factors of different supplementary cementitious materials is challenging. In reality, modern cements usually already contain various supplementary cementitious materials, rather than just ordinary Portland cement. Therefore, it is of importance to be aware of the exact composition of the cement used, and to judge its reactivity accurately as well as evaluating the necessity and effects of adding even more SCMs.

In order to estimate the absolute cementitious content of a concrete mixtures a correction factor, as mentioned earlier the k-value, based on the reactivity of each SCM is often utilized. The k-values should however only be taken as indicative values since variations in composition of SCMs exist, leading to geographically varying values depending on code or guideline used. In this thesis the k-values given in the Eurocodes will be used since they represent a good estimate for the SCMs available for the practical work done later. However, it is worth mentioning that the constant development and investigation of possible new SCMs means that research regarding reactivity of SCMs in different concrete compositions is an ongoing process. One thing is certain, the use of supplementary cementitious materials will continue increasing, both for making more durable concretes but mainly for reducing the CO_2 emissions produced by the concrete industry (Snellings, 2016).

Another increasingly popular type of concrete is the self-compacting or self-consolidating concrete (SCC). This type of concrete might also be considered as a high performing concrete on its own. The SCCs highly flowable nature without occurring segregation, makes it ideal for concrete structures where compacting of regular concrete would be difficult or even impossible. The negligible need of compaction and vibration when using SCC also leads to notable savings in labour. It is therefore reasonable to believe that a self-consolidation concrete with a high compressive strength would be highly beneficial. As Naik et al. demonstrated in 2012, it is possible to make SCC with 28-day compressive strengths of 62 MPa even by replacing 35 % of the cement with fly ash. This making the concrete not only advantageous economically, material and labour-wise, but also favourable from a sustainability aspect. However, it must be mentioned that the relatively high use of fly ash has a considerable negative impact on the short term strength development of the concrete (Naik *et al.*, 2012). The extensive use of fly ash is by no means obligatory when producing SCC, other mix designs to achieve quicker and higher strength concretes are possible, but likely at the expense of material costs.

Similarly, as for regular high-strength concrete, the adding of silica fume is also an effective way of increasing the compressive strength of SCC. This was demonstrated at the construction of the World Trade Center in San Marino, where a high strength concrete with properties otherwise similar to a self-consolidating concrete was required. The main requirements for this concrete included a high fluidity, slump flow ≥ 600 mm even 1 hour after mixing, and both a high early as well as final compressive strength with 1-day cube compressive strength ≥ 40 MPa and at 28 days ≥ 80 MPa. Furthermore, it had to exhibit a low drying shrinkage, ≤ 500 μm/m after 2 months, a dynamic elastic modulus over 50 GPa and uniformity of material properties when core samples of the field tests were examined (Collepardi *et al.*, 2003). The composition and performance of the resulting concrete can be seen in Table 2.2.

Table 2.2. Composition and performance of the self-consolidating high-strength concrete used at the World Trade Center in San Marino (Collepardi *et al.*, 2003).

Portland Cement (CEM I 42.5 R)		*465 kg/m^3*
SILICA FUME		*65 kg/m^3*
WATER		*175 kg/m^3*
GRAVEL (15-22 mm)		*195 kg/m^3*
GRAVEL (6-15 mm)		*720 kg/m^3*
SAND (0-6 mm)		*710 kg/m^3*
*ACRYLIC SUPERPLASTICIZER**		*4.6 kg/m^3*
water/(cement+silica fume)		*0.33 kg/m^3*
Slump flow at 5 and 60 min. (mm) AT 20°C		*730-600*
Compressive Strength (MPa) at:	*1 day*	*50*
	28 days	*95*
Drying shrinkage at 60 days (μm\m)		*380*
Dynamic elasatic modulus (GPa) at 28 days		*45*

** Dynamon SR1, Mapei, Italy*

The exceptional uniformity generally connected with even uncompacted SCC, due to its high resistance to segregation and bleeding, was also confirmed for the above-mentioned concrete by testing of core drillings from an uncompacted 1500 mm thick concrete sample. These core tests showing uniform results of compressive strength, dynamic elastic modulus and specific gravity throughout the whole thickness of the sample proves that also high-strength concrete can be produced and used in structures similarly to self-consolidating concrete *(Collepardi* et al.*, 2003)*.

The high-strength concrete, or perhaps more generally speaking, high-performance concrete, can usually be considered as a custom-made concrete designed according to project-specific requirements. These requirements demanding the use of HPC ranges from the simple need of high compressive strength, to durable concrete that at the same time must be economically viable. However, it is safe to assume that an HPC designed to minimize the cross-section of highly loaded column, is not also suitable for a massive concrete structure exposed to an aggressive marine environment that must be highly durable. Even though the HPC concrete used for columns might also provide the necessary properties, such as low permeability, needed for making durable marine structures, other factors must also be considered in such different cases. The aspect of considerable heat development connected with HPC might for instance not be a problem in the case of relatively small columns. But if the same HPC would be used in the case of a massive concrete structure, the heat development can cause concern during the curing, and therefore this aspect must be carefully investigated in the design process of the concrete mixture. However, in both cases the use of conventional concrete would not suffice, or certainly not be efficient. This shows that developing a universal high-performance concrete can prove difficult and that providing pre-designed HPC mixes might not even be profitable from the perspective of a concrete producer. On the other hand, developing and producing high-performance concrete, particularly higher strength concretes of standardized strength classes, and promoting them might encourage designers in utilizing this, sometimes underestimated, material.

2.3 Use of high-strength concrete

2.3.1 In Europe

The widespread use of high-strength concrete in Europe is a relatively new practise. This is mainly because that for a long time higher strength concretes were not specifically regulated. Therefore, the use of HSC required extensive tests to attain, often project-specific, certificates ensuring the performance of that explicit concrete. The quantities of HSC used in construction projects has generally amounted to only a small part of the total concrete used. Thus, the additional, relatively expensive, testing procedures needed for using HSC were usually thought of as an unnecessary extravagance (Breitenbücher, 1998). In Germany this changed in 1995 with the introduction of the “Guidelines for High Strength Concrete” by the German Association for Reinforced Concrete, DAfStb. These guidelines dealing with concretes in the range of C55-C115 acted as a supplement to the existing German guidelines at the time that only dealt with concretes up to C55 and meant that the expensive project-specific testing of HSC was not required anymore. The same phenomenon also occurred around other European countries at that time, e.g. in Britain where the technical report 49 “Design guidance for high strength concrete” was published in 1998 as an extension to the British Standard 8110 which only dealt with concrete up to 40 MPa. Later, the introduction of the Eurocodes has facilitated the use of HSC throughout Europe. It is also worth noting that Norway was the first country to include high-strength concrete with a characteristic cube strength of up to 105 MPa in its national design standards, NS 3473, as early as 1989 (Helland, 1997.

Before any specific standards or guidelines concerning specifically high-strength concrete, the use of this material required extensive testing for acquiring permits that recognized the improved properties of high strength concrete. Such was the case when HSC was used in

Germany for the first time in 1990 for the 186 m high-rise building of the Trianon in Frankfurt, Figure 2.1.

Figure 2.1. The Trianon in Frankfurt constructed 1990-1993 was the first high-rise building incorporating HSC in Germany (structurae, 2019a).

Because concrete with a strength class of C85 was going to be used in critical load bearing columns and walls for the Trianon, permits concerning the structural capabilities of this structural concrete had to be attained separately (Breitenbücher, 1998). The Trianon was successfully constructed and since then HSC has received a broader recognition and acceptance as a suitable building material also in the building sector.

Although the tallest buildings in Europe are not of very remarkable heights compared to the tallest buildings in the world, the number of skyscrapers constructed in Europe in recent years is still substantial. The overall trend of using HSC in tall buildings also correlates with the use of this material in European construction projects. High-strength concrete can be found at least in some form in most of the skyscrapers in Europe, ranging from plain cast-in-situ HSC or steel composite load bearing members to pressed concrete floor slabs and decorative precast façade elements.

Norway is considered to be among the pioneering countries to utilize high-strength concrete on a large scale, not only in Europe but also in the world. The long use of superplasticizers and early availability of silica fume were both contributing factors in Norway's early experimentation with HSC. But the foremost reason for developing HSC was the urgent need for a durable construction material that could compete with the expensive steel as the main building material for the offshore structures constructed for the oil and gas industry. Since the early uses of HSC at the offshore structures, the HSC has also been utilized further at a large extent around highway structures in Norway. Numerous bridges like the Helgeland bridge, shown in Figure 2.2, has been constructed in Norway since. At the opening of the Helgeland Bridge in 1991, after two years of construction in challenging conditions, the 425 m main span of the bridge was the third longest concrete main span in the world. The cable stayed bride is made by cast in place prestressed concrete with a compressive strength of 65 MPa used throughout (structurae, 2019b). High strength concrete has continued to be used for bridges in Europe ever since, especially in prestressed and composite structures where higher strength concrete is very useful for achieving longer spans and more slender profiles.

Figure 2.2. The cable stayed Helgeland Bridge in Norway (structurae, 2019b).

2.3.2 Around the world

Concrete, and high-performance concrete in particular, has played a major part in facilitating the construction of taller and generally more extraordinary buildings, but it has also had to overcome challenges risen from the trend of building higher and higher. Some success-stories of world renown building projects and how HSC was used at the projects will be presented in this chapter.

Firstly, as buildings become taller, the importance of reducing deformations and sway of the structure increases. Thus, just by using high-strength concrete instead of normal-strength concrete in load-bearing elements, the stiffness of the building is improved considerably. In fact, the high elastic modulus of HSC should be considered equally important, if not even more important than the compressive strength of HSC in the design of skyscrapers. An example of this is the Two Union Square building in Seattle, where HSC with a compressive strength of 130 MPa was cast in 3 m wide steel tubes to provide sufficient stiffness to the building. In this case the remarkably high compressive strength was only a side effect from reaching the high elastic modulus required, 50 GPa, to make the building stiff enough, strength-wise a 90 MPa concrete would have sufficed (Aïtcin, 1998). Composite structural elements, such as the columns in the Two Union Square building, and generally combining structural steel and concrete is a very useful practise when designing and constructing very tall buildings today. Not only are the commonly used hollow steel sections filled with HSC efficiently increasing the load bearing capacity and stiffness of the structure as well as making it more ductile, it is also an efficient way to reduce on-site work and keeping the dead weight of the building reasonable.

The Taipei 101 tower in Taiwan provides a good example of how high-performance concrete can be used efficiently in combination with structural steel sections. Here the so-called super-columns, as shown in Figure 2.3, are regarded as the most important structural elements. These composite super-columns are steel box sections filled with 70 MPa HSC and are located at the perimeter of each floor of the building. In addition to the 8 super-columns per floor, smaller but similarly concrete filled steel box columns make up the core of the building. For the 62nd floor and below the steel box columns were filled with concrete, and above this they were left hollow to minimize the gravity load of the building (Büyüköztürk and Lau, 2004).

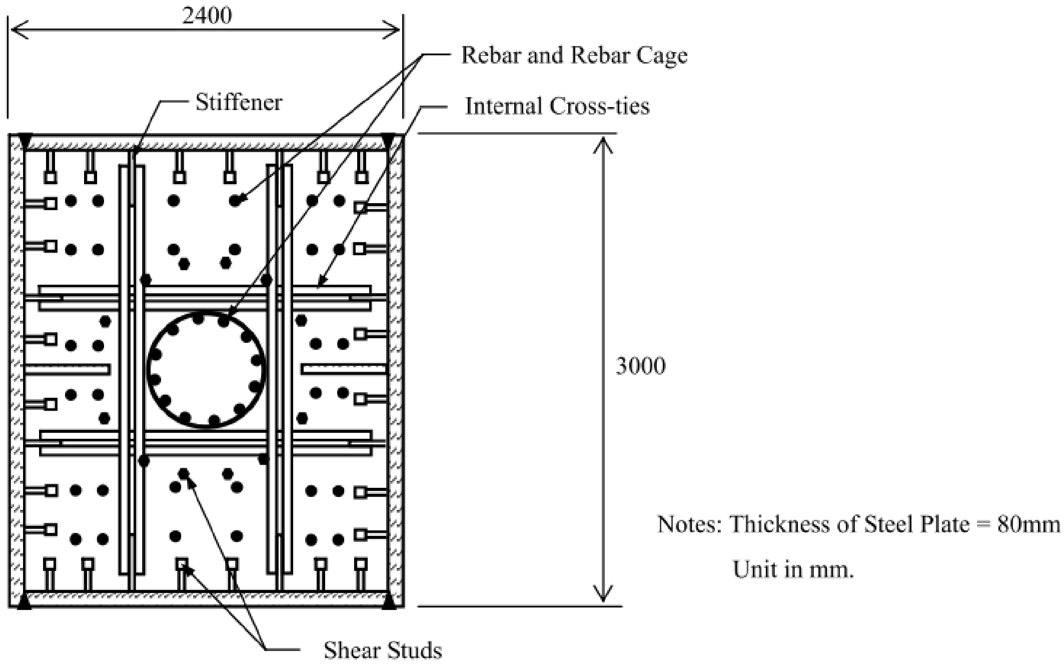


Figure 2.3. Cross-section of typical super-column in the Taipei 101 Tower. (Shieh et al., 2003)

The concreting of the composite columns for the Taipei 101 Tower was executed by pumping the HSC from the bottom of the columns up. By this approach air can be assumed to avoid being trapped in the heavily reinforced column as long as the concrete has a high flowability. To assure proper placement of the concrete at the Taipei 101 project, the concrete used was designed to have a slump and slump flow of 25 ± 2 cm and 60 ± 10 cm, respectively. Extensive testing was conducted prior to the actual casting to ensure the proper behaviour of the concrete design. In addition to previously mentioned properties, the concrete was also designed to minimize creep and shrinkage effects, as well as not allowing bleeding segregation (Büyüköztürk and Lau, 2004).

The issue of maintaining adequate pumpability while also avoiding bleeding and segregation can become a real challenge as the concrete has to be pumped to high altitudes and placed in densely reinforced spaces. This became evident especially clearly at the construction of the Burj Khalifa in Dubai, Figure 2.4a, which when finished in 2010 became the tallest building in the world. However, the difficulties were overcome without major problems by careful attention to pre-construction testing of the materials and equipment as well as constant quality controls during the construction stages. These precautions and arrangements are discussed more thoroughly in Chapter 3.2.1.

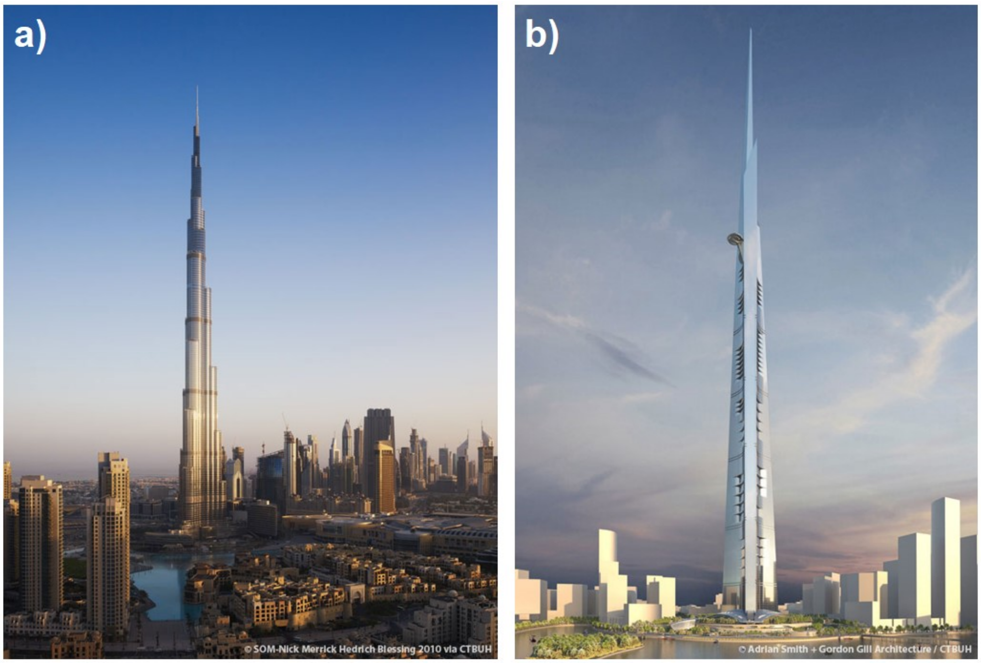


Figure 2.4. The Burj Khalifa in Dubai and b) a rendering of the completed Jeddah Tower in Saudi Arabia, both structures primarily made with concrete. (CTBUH, 2019)

The successful use of high-strength concrete in the construction of the Burj Khalifa has encouraged the further use of HSC in most modern super high-rise buildings, including the Jeddah Tower in Saudi Arabia, Figure 2.4b, which is under construction and will be the tallest building when completed. The record-breaking skyscraper which is expected to be complete in late 2019 and reach an altitude of over 1 km uses HSC in piles, foundation, columns and core elements of the building (Nehdi 2013).

2.4 Guidelines and codes connected to high-strength concrete

2.4.1 In Europe

Naturally, the guidelines followed in Europe concerning structural design and construction in general consists of the Eurocode standards, and more specifically concerning concrete, the Eurocode 2 (EN 1992-1-1, 2004) accompanied by the material specification standards EN 206. Even though the Eurocodes allow for the design of concretes up to 105 MPa, no specific definition regarding high-strength concrete is given in the standards. However, it can be noted that formulas, e.g. for calculating the stress distribution, change for concretes possessing compressive strengths over 50 MPa, this indicating some distinction between NSC and HSC. Even so, the behaviour concrete is thought to change considerably after reaching a certain compressive strength, around 50 MPa, that simply extrapolating formulas that are applicable on normal-strength concrete is not good enough for adequately designing higher strength concrete (Mendis, 2003; Mcfarlane, 2007). These differences distinguishing HSC from NSC includes increased brittleness, poorer fire resistance and a debated sufficient frost resistance without additional air-entrainment. The fact that the Eurocodes does not differentiate further between NSC and HSC has led to the development of independent guidelines, such as the Concrete Society's Technical Report 49 "Design guidance for higher strength concrete", as well as multiple publications by fib and RILEM explicitly dealing with various aspects concerning the design of HSC structures.

The behaviour of concrete under elevated temperatures has been studied since the 1950s and is by now well understood. The fact that HSC behaves poorer at elevated temperatures than NSC has been established much later. It was when Phan and Carino (2003) compared test results of HSC columns in fire situations with relevant codes at that time, it became clear that those codes generally overestimated the strength of HSC members subjected to elevated temperatures. Since then it has become evident that HSC, compared to NSC, suffer higher strength loss, particularly in the intermediate temperature range, as well as more explosive spalling when exposed to the same elevated heating conditions (Phan and Carino, 2003). This has since been recognized and additional fire safety measures are now included in most guidelines concerning HSC, including the Eurocodes (EN 1992-1-2, 2004).

Guidelines concerning the mixture design of high-strength concrete are often either too vague or too specific to be of any actual use. Since even small variations in local raw materials, such as aggregates or admixtures, may cause significant changes in the behaviour of the fresh concrete, the need of trial batches are generally needed when designing new HSC mixtures. Therefore, the proportioning of HSC is typically trial and error-based, which has led to the concrete producers developing their own methods for perfecting the HSC mix depending on relevant parameters and requirements. However, some basic proportioning methods exists, such as the Finnish method "Korkealujuuksisten betonien suhteitus" by Penttala (1990) and the American Concrete Institutions "Guide for Selecting Proportions for High-Strength Concrete with Portland Cement and Fly Ash" (ACI 1993). These basic proportioning methods can be useful when making initial trial batches, but since materials and their availability are constantly changing, the methods quickly become outdated and are rarely consistent when different materials and admixtures are used.

2.4.2 Around the world

There are many esteemed guidelines originating from different parts of the world that provides support in the design of high-strength concrete structures. One of the most influential and valued providers of guides and reports concerning explicitly HSC is the American Concrete Institute (ACI). The ACI offers guides and reports concerning everything from proportioning to quality control of HSC. Another corresponding organization to the ACI is the

International Federation for Structural Concrete, fib, which provides similar guidelines regarding design and production of concrete structures, the most relevant being the extensive "fib Model Code for Concrete Structures 2010".

Furthermore, the existence of national guidelines such as Japans "Recommendation for practise of High-Strength Concrete" (2005) and Singapore's "Design guide of High Strength Concrete" (2008) provides useful recommendations regarding local concreting practises adapted for HSC. These national guidelines are not only useful for the country in question but can also be utilized in countries with similar conditions and practises but lacking own codes regarding HSC. Alternatively, the necessary specifications concerning higher strength concretes can be integrated in the general building codes, such as in the Canadian "Design of Concrete Structures" (CSA-A23.3-04) and the Australian "Concrete Structures" (AS3600-2009).

However, as illustrated by Wu et.al. (2010), the recommended design values of certain fundamental engineering properties may vary considerably depending on which design code is being followed. An example of this is shown in Figure 2.5 where the recommended values according to American, Australian and European design codes for the characteristic flexural tensile strength, f'_{cf}, and the characteristic principal tensile strength, f'_{ct}, are plotted against the compressive strength, f'_{c}, of concrete. The values plotted for f'_{cf} and f'_{ct} are according to the standards and corresponding equations shown in Table 2.3.

Table 2.3. Table showing the equations used in various standards to calculate the recommended values of the characteristic flexural tensile strength, f'_{cf}, and the characteristic principal tensile strength f'_{ct} (Adapted from Wu et al., 2010)

Standard	**Recommendations for f'_{cf} and f'_{ct}**
AS3600-2001	$f'_{cf} = 0.6\sqrt{f'_c}$ (1) $f'_{ct} = 0.4\sqrt{f'_c}$ (2)
AS3600-2009	$f'_{cf} = 0.6\sqrt{f'_c}$ (3) $f'_{ct} = 0.36\sqrt{f'_c}$ (4)
ACI 318-2005 ACI 363R	$f'_{cf} = 0.62\sqrt{f'_c}$ (5) $f'_{ct} = 0.59\sqrt{f'_c}$ (6)
Eurocode EC2-2004	$f'_{ct} = 2.12\ln\left[1 + \frac{f'_c + 8}{10}\right]$ (7) $f'_{cf} = \max[(1.6 - h/1000)f'_{ct}; f'_{ct}]$ (8) Where h is the depth of the cross-section

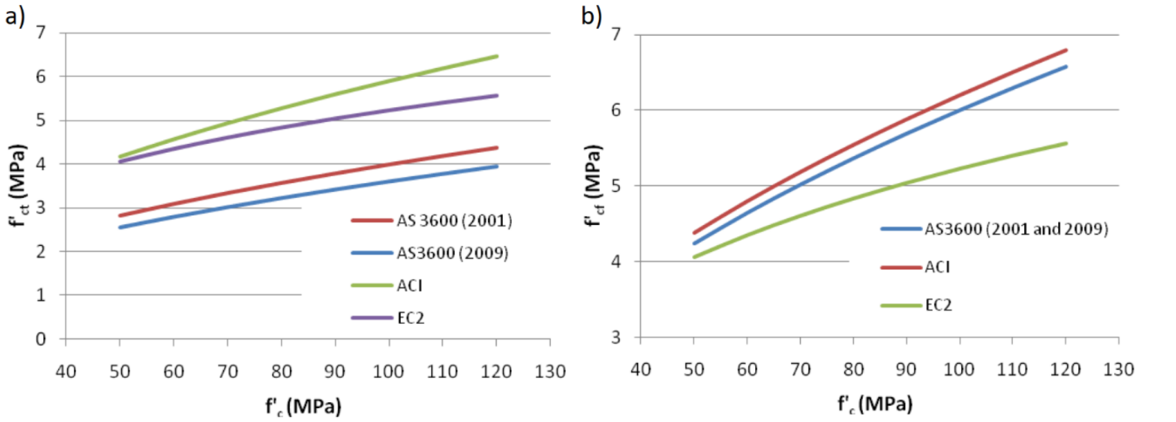


Figure 2.5. Comparison between different standards on recommended values for the characteristic flexural tensile strength, f'_{cf}, and characteristic principal tensile strength, f'_{ct}, plotted against the concretes compressive strength (Wu et al., 2010).

In addition to the national standards and building codes, researchers are also constantly investigating and refining equations that may represent the behaviour of high-strength concrete more accurately. The modulus of elasticity for concrete, E_c, and especially how it differs between normal and high-strength concrete is a good example of an engineering property that has been thoroughly studied. For example, Setunge (1993) showed that Equation 9, used in the old Australian building code, AS-3600 2001, tended to overestimate the modulus of elasticity for concrete with compressive strength above 40 MPa. Thus, a new equation for calculating the modulus of elasticity for HSC more accurately was introduced in the renewed AS-3600 2009, Equation 10, while the old equation was still conserved for calculating the elastic modulus of NSC. Examples of equations for calculating the elastic modulus are presented in Table 2.4 and plotted against each other in Figure 2.6 to illustrate how much these values may vary depending on which standard is followed (Wu et al., 2010).

Table 2.4. Examples of proposed equations for calculating the elastic modulus of concrete, E_c, from both standards and researchers (Adapted from Wu et al., 2010).

Standard	**Recommendations for E_c***
AS3600-2001	$E_c = 0.043\rho^{1.5}\sqrt{f'_c}\ \pm 20\%$ (9)
AS3600-2009	$E_c = \rho^{1.5}(0.024\sqrt{f'_c} + 0.12)\ \pm 20\%$ (10)
ACI 318-2005	$E_c = \left(3.32\sqrt{f'_c} + 6895\right)\left(\frac{\rho}{2320}\right)^{1.5}$ (11)
Eurocode EC2-2004	$E_c = 22\times 10^3\left(\frac{f'_c}{10}\right)^{0.3}$ (12)
Mendis et al. (1997)	$E_c = 0.43\eta\rho^{1.5}\sqrt{f'_c}\ \pm 20\%$ where, $\eta = 1.1\text{-}0.002f'_c \leq 1.0$, (13)
Carrasquillo et al. (1981)	$E_c = \left(3320\sqrt{f'_c} + 6900\right)\left(\frac{\rho_c}{2320}\right)^{1.5}$ (14)

* The nominal density of normal weight HSC $\rho = 2400$ kg/m^3

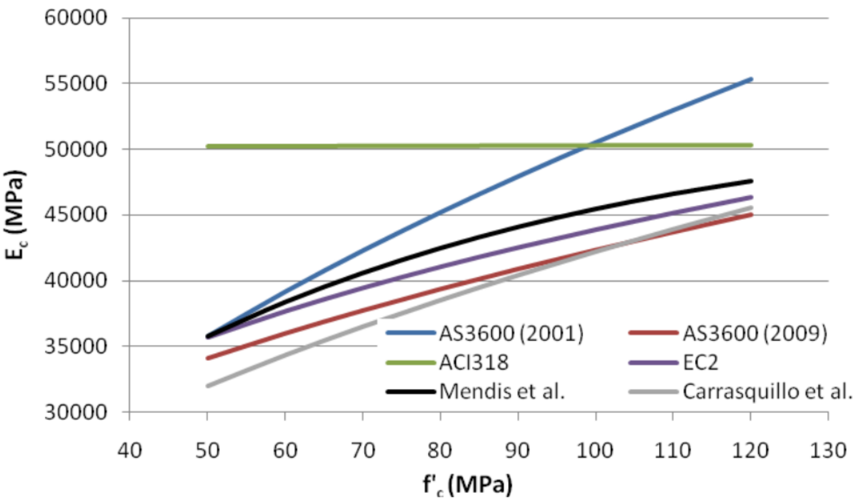


Figure 2.6. Varying values of the elastic modulus, E_c, plotted according to the equations presented in Table 2.4 (Wu et al., 2010).

3 Considerations for producing and optimizing the use of high-strength concrete

3.1 Production of high-strength concrete

3.1.1 Structural design considerations

One of the most significant differences between high-strength concrete and normal-strength concrete from a structural point of view, besides the obvious compressive strength, is the increased brittleness of HSC. This can be understood by looking at the different behaviours in the stress-strain relationships between concretes of various strength classes illustrated in Figure 3.1. This decrease in ductility for higher strength concretes can be explained by the delayed spreading of internal microcracks, which in lower strength concretes spreads extensively at a relatively low stress level to form an interconnected network that helps redistribute the energy and thus making the material more ductile (Mendis, 2003). Still, the delayed forming of microcracks also mean that the higher strength concretes maintain an elastic behaviour longer. Another important aspect that should be recognized is the decrease of ultimate compressive strain as the strength increases. However, once the undesirable behaviours are acknowledged there are numbers of solutions to remedy them so that sudden failures can be avoided.

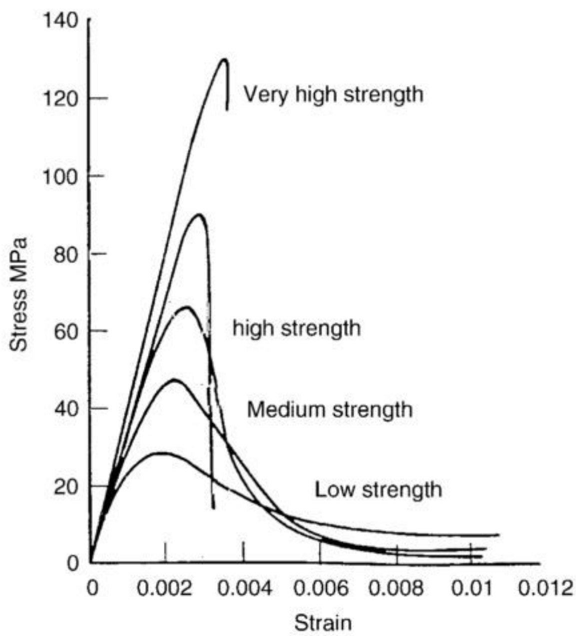


Figure 3.1. Typical stress-strain relationship for concrete under uniaxial loading (Mendis 2003)

For columns made of HSC sudden failure can easiest be prevented by sufficient confinement of the concrete core. This confinement pressure is usually provided by the reinforcement links perpendicular to the column length. Below are shown examples of a poorly confined, Figure 3.2a, and a well confined RC column, Figure 3.2b (Mcfarlane, 2007). However, the needed confinement pressure can also be attained using so-called composite columns where the concrete is cast inside hollow steel sections which both acts as mould for the column, but also increases the total capacity of the column.

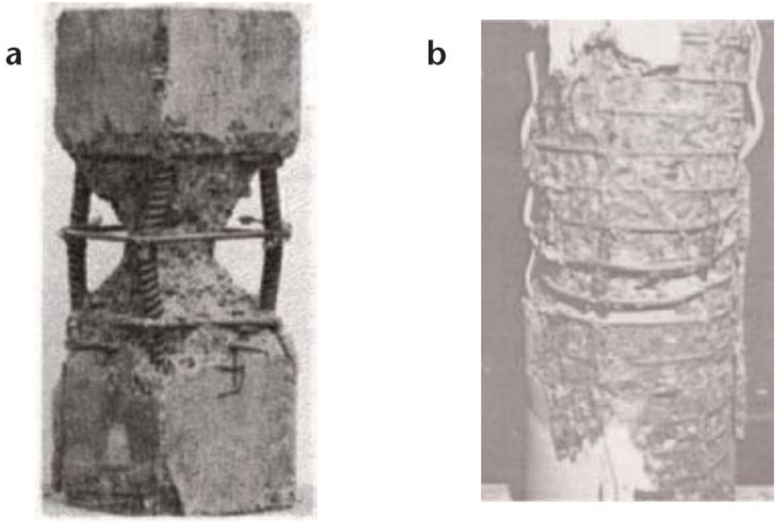


Figure 3.2. Examples of a), transverse reinforcement links poorly confining the column core and b) properly confined column core by adequate transverse reinforcement (Mcfarlane, 2007).

Because of the brittle nature of the high-strength concrete, it is mostly used for simple structural members under compression, such as columns. However, HSC can also be applied for flexural- and shear-stressed members such as beams and walls. This use of HSC is particularly useful when minimizing the dimensions of bridge girders as well as when all-concrete building frames are called for. When HSC was used for these kind of stresses in Germany for the first time, it had to be ensured the brittleness of the material would not be a problem (Breitenbücher, 1998). The concrete with a strength class of C105 to be used for the frame structure of the high-rise building "Taunustor" in Frankfurt in the middle of the 1990s therefore had to be thoroughly tested. Since HSC was going to be used in the building for lower level columns it was thought that the same concrete could be utilized for the frame structure around big window openings as well. The large-scale testing of the relevant sections showed that, despite the brittleness of HSC, the failure of the structure was still preceded by extensive cracking and plastification of the reinforcement steel. Thus, demonstrating the suitability of the material also for members subject to bending moments and shear forces.

Another way to make concrete more ductile is by adding fibres, most commonly steel or synthetic fibres, into the concrete mix (Singh, 2017). In other words, making so called fibre-reinforced concrete (FRC), or if the bulk concrete is of higher strength, fibre-reinforced high-performance concrete (FRHPC). The fibres introduced into the concrete matrix prevents the microcracks present in both the cement paste and the interfacial transition zone (ITZ) from rapidly expanding when come under tension. The improved stress-strain behaviour of fibre reinforced HSC and NSC compared to plain concrete can be seen in Figure 3.3.

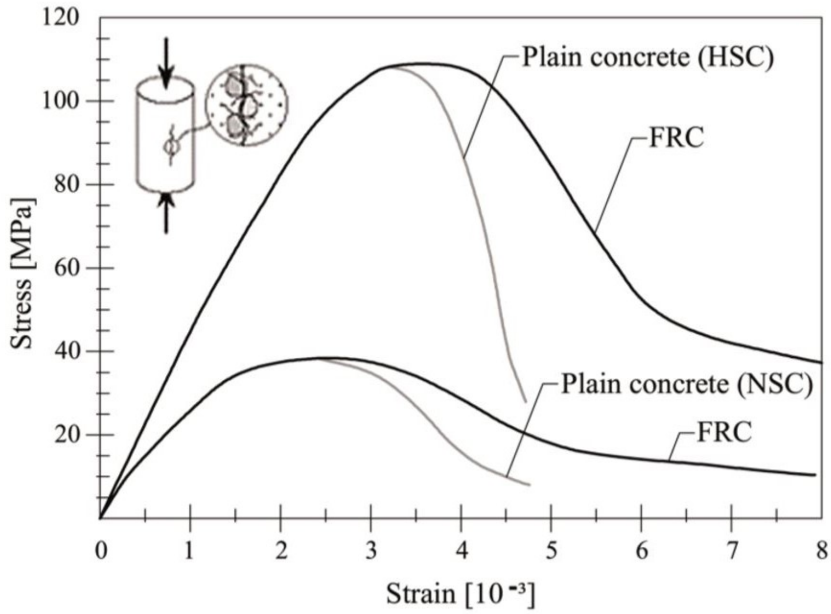


Figure 3.3. Main differences between plain and fibre reinforced concrete under uniaxial compression (fib, 2012)

Furthermore, the introduction of fibres into HPC have additional advantages than only improved ductility, which is the case for regular strength concrete reinforced with fibres. The FRHPC also shows a significant strain hardening behaviour which leads to both improved strength and toughness of the concrete (Büyüköztürk and Lau, 2004).

3.1.2 Mix design

The mix design of high-strength concrete, despite utilizing the same raw materials used for regular concrete, can be fairly complex and usually requires a trial and error method for optimizing the mixture. Even though HSC may consist of the same basic materials as NSC, the raw material used for producing HSC must be of highest quality and combined in the optimum proportions (Rashid and Mansur, 2009). Although many methods for mix design of HSC exists, small differences in aggregates or admixtures due to local availability can cause significant variations concerning the rheological properties of the fresh concrete mixture. Therefore, it is often crucial to make trial batches to verify that the required aspects for a successful cast are fulfilled. The fact that most HPC mixtures are case specific with unique requirements to be fulfilled should mean that the mix design process is to be initiated well ahead of the delivery of the concrete to the work site. However, this is not always the case, meaning that the actual strength, which is determined from the 28-day strength test of the trial batch, is yet unknown when the actual concrete batch is delivered and cast (de Larrard and Sedran, 2002).

As shown by Alves et al. (2004), considerable differences between high-strength concrete mix proportioning methods exist, both concerning performance of the hardened concrete, but most notably for the material consumption. Perhaps most importantly, the use of cement could be reduced by 50% when certain mix proportion methods were used. It is therefore possible to choose whether to focus on maximizing the performance of the concrete or minimizing the material consumption. However, this mainly indicates that whatever the requirements of the concrete are, it is generally possible to optimize the existing mix proportioning methods for specific cases.

The influence on the workability of the fresh concrete mixture is another important aspect that should be considered in the mix design stage. The selection on supplementary cementitious materials and chemical admixtures, as well their proportions and time of addition to the concrete mix, can have a considerable effect on setting times and workability. This becomes of great importance as the distances that the ready-mix concretes must be transported increases, especially for low w/cm ratio concretes such as HSC. These issues, along with recommendations for preventing them, will be discussed more in detail in the next chapter.

3.1.3 Casting and workability

One of the, perhaps unjust, beliefs associated with using high-strength concrete is a more demanding casting process. More demanding in the sense that the fresh high-strength concrete might require more work to cast properly and extra care to avoid any considerable loss of workability, or so-called slump loss. This is of course due to the lower w/cm ratios and an extensive use of water-reducers or superplasticisers (SP), which behaviour might not always be fully understood.

It has been shown that in addition to using superplasticisers based on different compounds (Chandra and Björnström, 2002), also the dosage and time of the addition of SP can influence the slump loss of the concrete mixture (Punkki et al., 1996). To avoid sudden slump losses, it is crucial to predict the SPs behaviour in the concrete mixture correctly. By doing so the concrete is assured to maintain sufficient workability throughout the casting without any need of retempering.

Retempering, the adding of extra water and/or chemical admixtures when the concrete arrives to the construction site, can be an effective way to prevent slump loss. However, the adding of extra water should be avoided since it has a negative effect on the strength of the hardened concrete due to the altered w/cm ratio. Naturally then, if retempering is indeed unavoidable, it should be done by adding chemical admixtures such as a superplasticiser instead of plain water. Retampering with a chemical admixture, compared to with water, requires a more carefully considered dosage but will have a considerably less damaging effect on the strength of the hardened concrete (Erdoğdu, 2005). It should also be made clear that especially when high-strength concrete is used, it is of utmost importance that the design strength of the concrete is truly reached. Therefore, instructions given by designers and concrete suppliers regarding the handling of the concrete to reach the desired properties of the hardened concrete must be strictly followed. However, if no mention regarding retepering is stated by the designer or supplier of the concrete it should not be ventured upon. Similarly, instructions regarding the curing of concrete should be followed thoroughly.

Another factor influencing the workability of the concrete is the proportions of supplementary cementitious materials possibly present in the concrete mix. Johari et al. (2011) showed that the general use of fly ash (FA), silica fume (SF) and ground granulated blast-furnace slag, GGBFS, improves the workability, whereas the incorporation of metakaolin significantly reduces the workability. However, it was also shown that unlike the use of FA and GGBFS which continued to have a positive effect on the workability as the dosage increased, the optimum dosage of SF for improved workability seem to be between 5-10 % replacement of the cement content. In contrast to Joharis et al. (2011) studies in which only up to 15 % of the cement was replaced by SCMs, other studies indicate that larger dosages of especially FA and GGBFS can reduce the slump loss of the concrete mix. Erdoğdu et al. (2011) showed that concrete mixes containing 30 % and 20 % FA replacement of cement decreased the slump loss compared to the concrete containing no SCMs. Notably is that the concrete mix containing only 7 % FA slightly increased the slump loss. Similarly, Marushima et al. (1993) found that replacing 40 % of the cement with very fine GGBFS slightly improved the workability but also reduced the rate of slump loss.

The suitability and effects of the available equipment for mixing, transporting and pumping or casting of the concrete should also be evaluated to efficiently produce and supply workable HSC. As reported by Carbonari et al. (2003), it is important to optimize the mixing time in accordance to the mixer used and for the concrete in question. Furthermore, the pumping equipment should be verified to be able to handle the concrete, and likewise the concrete should be able to be pumped with the available equipment without segregating or otherwise suffer from being pumped. The actual transportation of the concrete can also be taken advantage of. If the transportation distances are long, and if the transporting vehicles allow for it, the mixing water can even be added at a later stage after leaving the concrete factory to remain workable for longer after reaching the construction site. However, as HSC are very different, and even small variations in local materials and procedures have a large influence on the performance of the concrete, no universal rules can be applied to ensure a satisfactory behaviour for HSC in general. Therefore, testing and quality control whenever a new concrete is developed remains the only way of ensuring that the actual performance of the concrete fulfils the requirements.

3.1.4 Curing and hardening behaviour

The proper curing of concrete is vital, even very high-quality concrete can be spoiled by inadequate curing procedures. The importance of curing for high-strength concrete should therefore be emphasized in order to reach the full potential, or at least the required properties, of the hardened concrete. Especially when the HSC is designed for resisting severe conditions and aggressive environments the importance of curing becomes evident due to the impact it has on the permeability of concrete, and thus also the durability (Meeks and Carino, 1999).

It has also been recognized, that curing methods which would be sufficient for normal concrete might not be adequate for HSC (Aïtcin, 2003). This is mainly due to the different shrinkage behaviour shown by the two concrete types. Firstly, the most immediate threat of plastic shrinkage is more evident in HSC because of the scarce bleed water. However, this can be remedied by applying water curing or curing membranes immediately after the finish. Although this is recommended for both types of concretes, it is of greater importance for

HSC that appropriate curing practises are applied at once when the finishing of the concrete is done.

Secondly, while the governing long-term shrinkage of NSC is drying shrinkage, due to the comparably easily evaporable excess water, HSC is more likely to suffer from autogenous shrinkage due to self-desiccation (Aïtcin, 2003). Even though normal curing procedures such as curing membranes is adequate to prevent plastic shrinkage, it is not enough to prevent the occurrence of autogenous shrinkage in HSC. Providing external water in form of fog misting or water curing is therefore vital to keep the autogenous shrinkage at an acceptable level. Still, it is crucial that the external water is applied as soon as possible while the pores and capillaries of the concrete are still interconnected. Once the capillaries dry out and are disconnected from the surface, the external water source becomes insignificant since the water is unable to reach the inner pores and capillaries where the self-desiccation takes place. This problem of disconnection of capillaries might even be unavoidable for concretes with exceptionally low w/cm-ratios, in these cases internal curing (IC) might be needed to avoid autogenous shrinkage cracks (Zhutovsky and Kovler, 2012). IC can be achieved either by using pre-saturated porous lightweight aggregate or by adding super absorbent polymers into the concrete mixture. Both methods providing an internal source of free water to avoid self-desiccation during the cement hydration (Cusson and Hoogeveen, 2008).

It is also commonly recognized that in addition to the curing method used, also the curing temperature effects the strength development of concrete, especially at an early age. Adequate curing practises mainly ensures that the strength development is not hindered by the problems connected to the low water content of HSC described above (Meeks and Carino, 1999). Higher initial curing temperatures, around 30 °C and over, on the other hand are recognized to give the concrete a higher early strength. Thus, high-temperature curing can be used to accelerate the strength development of the concrete to enable quicker demoulding and generally fast-tracking construction. This also holds true for HSC. However, the differences in strength due to early high temperature curing gradually diminishes as the concrete matures and it is generally believed that concretes cured under more moderate temperatures, around 5-25 °C, catches up, or even surpasses, to the strength developed by high-temperature cured concrete (Yang et al 2015). Nevertheless, as every project is different, the main thing remains, also for HSC, to accurately predict the temperature development of the concrete keep the temperature within reasonable limits during the concrete curing as both too cold and too hot temperatures will harm the final performance of the hardened concrete.

3.1.5 Fire resistance

Considering that one of the most common application areas for high-strength concrete is indoors columns, the fire resistance becomes in many cases the governing and main durability concern. It is therefore unfortunate that HSC has shown poorer resistance to elevated temperatures and fires than normal-strength concrete (Kodur, 2005). However, the extent of the inferior fire resistance of HSC has been a debated extensively and even cases where HSC has performed well under elevated temperatures exist (Aïtcin, 2003). The consensus that can be made is that HSC generally has inferior fire resistance properties compared to conventional NSC. Both in the sense of higher strength losses with increasing temperature, shown in Figure 3.4, and an increased occurrence of fire induced spalling compared to NSC.

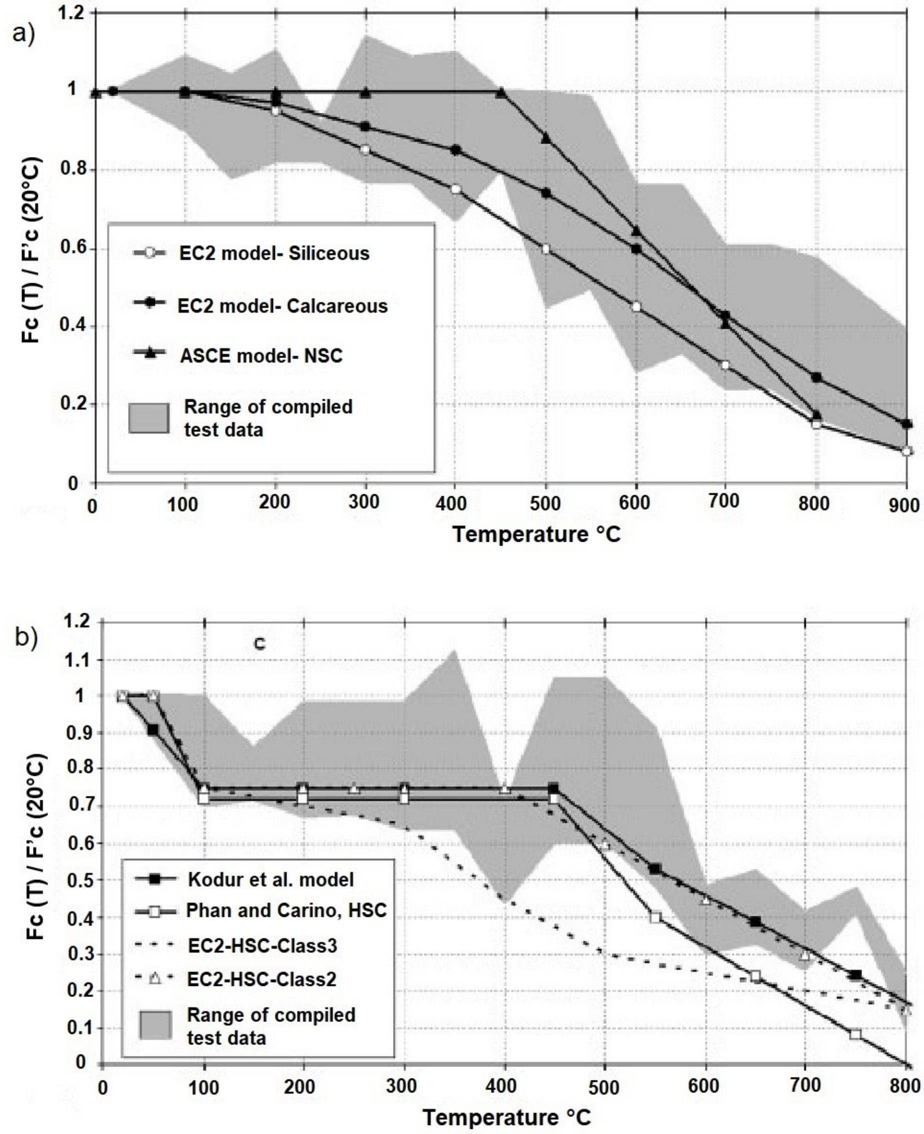


Figure 3.4. Strength changes at increasing temperatures for a) normal-strength concrete and b) high-strength concrete. (Kodur, 2008)

The dense and often almost impermeable microstructure of HSC makes it more vulnerable to fire induced spalling, and even explosive spalling, than normal-strength concrete. The more permeable NSC allows the growing pore pressure, due to the vaporising entrapped water heating up in the concrete, to escape. Whereas in the denser HSC, the entrapped water vapour and the resulting building pore pressure has no way to escape, leading to spalling of the concrete once the pore pressure reaches the tensile strength of the concrete (Akca and Zihnioğlu, 2013). It is therefore clear that the main reason for spalling of concrete is an adequate amount of water in the concrete, which is possible to vaporise. The amount of moisture that can be accepted without risk of spalling is according to Hertz (2003) 3 % in mass, this applying to normal concrete. However, denser concretes, such as HSC, might not be safe from spalling even with a moisture content of 0 %. This because even the chemically bound water released from hydrates dehydrating during increasing temperatures is sufficient to produce critical pore pressure in impermeable concrete.

Although high-strength concrete has shown to possess inferior fire resistance properties compared to normal-strength concrete, it can still be argued that it is considerably safer and easier to fireproof than steel members, which usually is the alternative to HSC when NSC is not sufficient. Particular cases where HSC structures are especially vulnerable are intense fires, resulting in a high rate of temperature rise, and when the concrete is very impermeable, i.e. when large quantities of silica fume has been used or the general strength of the concrete is very high, at least over 70 MPa (Kodur and Phan, 2007). The performance of high-strength concrete when exposed to fire can, and should in most cases, be improved in different ways in addition to general fireproofing methods for structural members such as increasing the protective cover of concrete or adding other insulating material.

One effective way to considerably improve the fire resistance of HSC is by the addition of polypropylene fibres (Kodur *et al.*, 2003; Akca and Zihnioğlu, 2013). The polypropylene fibres give the concrete a better fire resistance by minimizing the occurrence of fire-induced spalling. This is because the fibres mixed in the concrete melts at relatively low temperatures, about 160–170 °C, thus leading to the forming of pores through which the building pore pressure in a fire situation can escape. Another type of fibres that can increase the fire performance of HSC when included in the concrete mixture are steel fibres. The steel fibres simply improve the tensile strength of the concrete and thus reduce spalling by increasing the tensile strength of the concrete above the tensile stresses caused by the building internal pore pressure. However, the introduction of steel fibres to the concrete matrix may not fully eliminate fire induced spalling in HSC. Thus, adding a combination of both fibre types mentioned, i.e. hybrid fibres, would be the most effective way to increase the fire performance of high-strength concrete (Khaliq and Kodur, 2018). Additionally, increased air content through the addition of an air-entrainment agent in combination with PP fibres indicates to an improved spalling resistance for HSC exposed to fire (Akca and Zihnioğlu, 2013).

While the steel fibres improve the overall strength of the concrete, the addition of polypropylene fibres will slightly reduce the compressive strength of the concrete, both the initial strength (Breitenbücher, 1998) and the residual strength when subject to elevated temperatures (Akca and Zihnioğlu, 2013). This loss of strength can, and should be, counteracted by increasing the cement content of the concrete by approximately 25 kg/m^3 according to Breitenücher (1998). The recommended amounts of fibres to be added to the concrete mix for improved fire resistance is estimated by Kodur (2008) to be 0.1-0.15 % by volume of polypropylene fibres or about 1.75 % by mass of steel fibres.

The fire performance of high-strength concrete is also affected by structural parameters such as rebar placement, especially the tie arrangements, and the general size of the member (Kodur and Phan, 2007). For normally reinforced HSC columns exposed to building fires, a tie configuration with 135° hooks instead of 90° hooks, as can be seen in Figure 3.5, has been shown to effectively improve the overall fire resistance of columns.

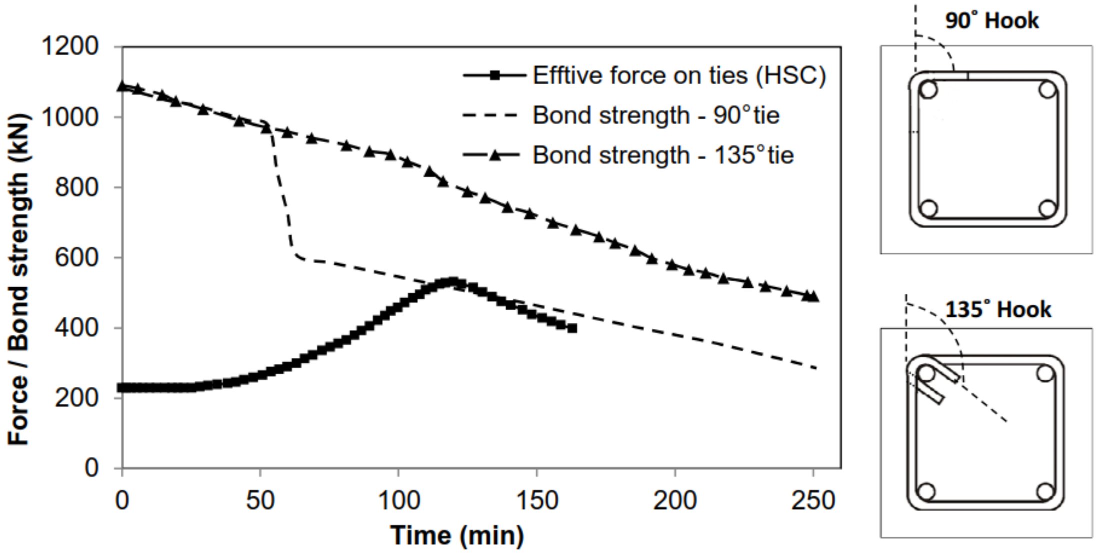


Figure 3.5. Comparison between different tie configurations and how they affect the bond strength in HSC columns under fire situation. (Khaliq, 2012)

The bond strength between ties and concrete becomes especially important in HSC columns where the low permeability of the concrete results in higher pore pressure that in turn causes higher forces applied on the confining tie reinforcements (Khaliq, 2012).

Additionally, standard precautions for improving fire performance of concrete members such as increasing member size and concrete cover, as well as reduced tie spacing for improved confinement naturally also applies for HSC.

3.1.6 Frost resistance

The frost resistance of concrete is often thought to increase when using high-strength concrete, or more specifically, when low w/cm-ratios are obtained. For normal-strength concrete exposed to freezing and thawing cycles, additional air-entraining is required. This additional air provides voids to where the free water in the concrete subject to freezing can expand. If no additional unoccupied air voids are available, the expanding water produces an internal hydraulic pressure that cannot be relived, thus causing tensile stresses in the concrete. In conditions where this mechanism can continue and be repeated, so-called freeze-thaw cycles, the tensile stress might eventually exceed the tensile capacity of the concrete and consequently cause cracking (Neville, 2011).

By using high-strength concrete, several of the factors causing concrete structures to be damaged by freeze-thaw cycles are reduced considerably, if not altogether erased Hale et al., 2009). Firstly, the tensile capacity of high-strength concrete is higher than normal-strength concrete. However, since the increase in tensile strength for HSC is not very significant and even the tensile strength of very high strength concrete remains relatively low, this aspect will not on its own improve the frost resistance of concrete notably. A more important factor in explaining why HSC, even without air-entraining, can be frost resistant is the low water content of the concrete. Due to the low w/cm ratios connected with HSC, the concrete simply does not contain sufficient freezable water to cause any significant tensile stresses in the concrete. The fact that all mixing water in HSC is generally consumed by the hydrating of cement, and thereby becoming chemically bound and non-freezable, is considered a central aspect of why HSC has an improved freeze-thaw resistance compared to NSC. Furthermore, the low permeability of a well-designed HSC effectively prevents external water to penetrate the concrete. The low permeability is therefore also absolutely crucial for making frost resistant low air-content concretes. It has been discussed if a limit w/cm ratio exists that would lead to adequate properties in making a frost resistant concrete without the need of additional air-entraining. Several studies have been carried out to show that high-strength concretes with w/cm ratios ranging from 0.24 to 0.36, and below, appears to possess satisfactory freeze-thaw resistance. However, Hale et al. (2009) mentions that similar concrete mixtures within the same range of w/cm ratios have shown unsatisfactory frost resistance. Thus, it can be concluded that freeze-thaw resistant concrete cannot be confidently realized simply by reducing the w/cm ratio.

While there also are other methods for making the concrete less permeable, such as using silica fume, the most important factor for successfully achieving an impermeable concrete remains good curing practises (Kukko and Tattari, 1995). Therefore, even though in theory frost resistant non-air entrained high-strength concrete might be possible to produce, air-entrainment is recommended if the concrete in question has not indeed been proven frost resistance. However, the required percentage of air content in the concrete can be expected to decrease as the w/cm ratio is lowered.

3.1.7 General durability

Although the dense microstructure of high-strength concrete might be the cause of some concern for the concrete durability in specific circumstances, such as in freezing environments and fire situations, it generally offers an advantage concerning the overall durability of the concrete structures.

Carbonation of concrete is the chemical reaction between atmospheric carbon dioxide, OH_2, and carbon hydroxide, $Ca(OH)_2$, in concrete to produce calcium carbonate, $CaCO_3$, according to the chemical reaction formula below:

$$CO_2 + Ca(OH)_2 \rightarrow CaCO_3 + H_2O \tag{15}$$

This reaction, starting from the surface of the concrete, causes the normal pH value of 13-14 of concrete to drop below 9. The normally high alkalinity of concrete is crucial for protecting the reinforcement steel from corrosion. Thus, the drop in alkalinity due to carbonation enables the corrosion of the embedded reinforcement steel once the carbonation has reached the concrete cover depth. This reaction is one of the main deterioration mechanisms of concrete today and often the cause of needed repairs of concrete structures. However, carbonation is largely affected by the diffusivity of concrete, which for high-strength concrete has been shown to be satisfactory low. Other deleterious phenomenon based on the ingression of harmful substances, such as chloride and sulphate ingression, are similarly connected to the diffusivity and permeability of concrete. Both diffusivity and permeability are closely connected to the porosity and pore structure of the concrete. The low porosity and favourable pore structure of HSC, mainly due to low w/cm ratios and often present supplementary cementitious materials such as silica fume, therefore minimizes the deleterious effects of the deterioration mechanisms mentioned above. The effect lower w/c ratios and addition of silica fume has on the diffusion coefficient can be seen in Figure 3.6a. Similarly, Khan and Lynsdale (2002) demonstrated the correlation between compressive strength and carbonation depth by combining previous studies (Byfors, 1985) with own results, as shown in Figure 3.6b. The concrete samples studied by Khan and Lynsdale (2002) were made using blended cement containing fuel ash and silica fume. The 100 mm cube specimens were stored for 2 years, similarly to Byfors's specimens that were stored 2 ½ years, in a normal atmospheric environment and at a constant temperature of 20 ± 3 °C and humidity of 65 ± 5 % RH.

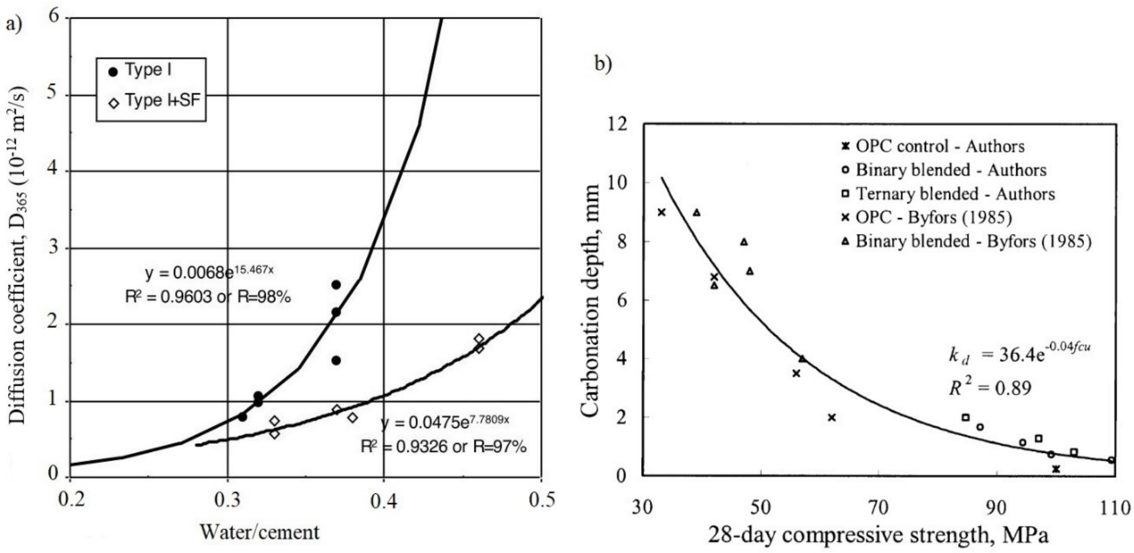


Figure 3.6. a) Correlation between the diffusion coefficient and w/c ratio (CCAA, 2009). *b) Effect of compressive strength on carbonation depth after 2 years, for specimens > 70 MPa, and 2 ½ years for specimens < 70 MPa* (Khan and Lynsdale, 2002).

Another aspect concerning the durability of concrete is whether high-strength concrete is subject to alkali-aggregate reactions (AAR), or more specifically, the most common AAR, alkali-silica reactions (ASR). This reaction occurs between the alkalies present in the cement pore solution and reactive mineral compositions found in some aggregate types. The result of this reaction is a water absorbing gel which expands and thus causing internal stresses that may lead to extensive cracking of the concrete (Marzouk and Langdon, 2003). Therefore, HSC with its low w/c ratio, low permeability and lack of free internal water might seem like an unlikely environment for the occurrence of ASR. However, Ferraris (1995) argues that in such dense and low air content concretes the lack of space for the expanding gel may cause concern. Ferraris (1995) also points out that water is always present at the start of hydration and that some reactions may occur when water is still available. Similarly to the freeze-thaw action, the tensile strength of HSC may arguably be sufficient for dealing with the internal stresses due to the expanding substances, but no real consensus on this matter concerning ASR has been reached either. However, it has been shown that HSC made with reactive aggregate is less affected by the negatives effects associated with ASR than normal-strength concrete (Trägårdh, 1994; Marzouk and Langdon, 2003). Marzouk et al. (2003) attributed the improved performance of HSC under favourable ASR conditions to the enhanced microstructure and the secondary pozzolanic reactions, in this case provided by silica fume, which is a known additive for preventing negative effects due to ASR (Farny and

Kerkhoff, 1997). Although Trägårdh (1994) observed some advantages by using HSC in preventing ASR, it could not be proven that HSC is completely unaffected by ASR. Thus, it is recommended that when reactive aggregates and high alkaline cement is used for producing high-strength concrete, the mix must be tested for deleterious alkali-silica reactions.

3.2 Optimizing the use of high-strength concrete

3.2.1 Application areas requiring high-strength concrete

As concrete is mostly recognized and used for its compressive strength, high-strength concrete is naturally made to enhance this material property even further. Even though steel is a structurally perfectly sound material for constructing high-rise buildings, making whole skyscrapers using steel has its problems. Not only is the steel highly vulnerable to fire, it has also been shown that structural steel frames are a costly alternative for high-rise buildings (Mills, 2010). Therefore, as concrete has seen a drastic rise in strength in recent decades, HSC has become a widely used and even favoured building material for constructing high-rise buildings. High-strength concrete is required for high-rise structures in the sense that normal-strength concrete is simply not adequate anymore, especially for the lower level highly loaded structural members.

Additionally, composite structures combining HSC and steel, at varying ratios depending on situation, can be considered the most optimal way of constructing high-rise buildings today. This way the undeniable benefit of using steel, which is reducing the total weight of the building, can also be exploited when building with concrete. However, as the clear majority of recent super-tall buildings (around 80 stories and more) constructed are primarily reinforced concrete structures, it can be understood that high-performance concrete in some form is the preferred material for constructing skyscrapers (Aldred, 2010). An indication that concrete is indeed at present the preferred skyscraper building material are that both the currently tallest building, the Burj Khalifa and the Jeddah Tower which will become the tallest building when completed, are made by HPC.

Before the construction of the Burj Khalifa, high-strength concrete was rarely pumped to high elevations. Normal-strength concrete on the other hand had been pumped to heights of several hundred meters in previous projects such as the earlier mentioned Taipei 101, the Petronas Towers in Kuala Lumpur and the Jin Mao Building in Shanghai. It is believed that the first pumping of HSC concrete to any significantly high altitude happened when SCC with a compressive strength of 110 MPa was pumped up to 200 m at the construction of the Trump International Hotel and Tower in Chicago which was completed in 2009 (Nehdi 2013). However, the single stage pumping of HSC up to an elevation of 568 m at the construction of the Burj Khalifa in 2008 crushed the previous world-record of concrete pumping height from 1994 when concrete with a compressive strength of merely 25 MPa had been pumped to 532 m at the Riva del Garda hydroelectric power plant.

Pumping of concrete, and especially HSC, to high elevations requires both carefully designed concrete mixtures and powerful pumps. Nevertheless, successful pumping of concrete is a vital part of maintaining the competitiveness of reinforced concrete as the main material for constructing high-rise buildings. Pumping failures or need to use cranes for placing large quantities of concrete means significantly reducing the casting rate and would thus question the viability of the material. For the Burj Khalifa project the pumping of the concrete represented a significant challenge and staged pumping was originally planned (Aldred, 2010). However, after extensive testing of pumping equipment and proposed concrete mixture to make sure pumping could be executed successfully in the demanding concreting conditions of Dubai single stage pumping was deemed possible. Both the C80 and C60 concretes that were used in the Burj Khalifa were considered highly workable HPC with a slump flow of around 600 mm. The effects of pumping the concrete at high pressures for a considerable time to the required heights were considered in the project and samples were taken regularly of all concretes when reaching the intended elevation. Although the pumping of the concrete on average resulted in a temperature rise of 2-3 °C, 10% reduced slump flow and roughly halve the plastic viscosity of the concrete, these effects were considered acceptable. To minimize the risks for pumping blockages and other problems, all pumping was conducted during night time and ice flakes were used as mixing water to keep the temperature of the concrete at an acceptable level. Naturally, as the building rose, and the pumping heights increased, lower grade concrete with higher w/cm ratios as well as smaller maximum aggregates could be applied, thus reducing the needed pumping pressure. The effect of the reduced concrete strength classes and maximum aggregate size had on the pumping pressure can be seen in Figure 3.7.

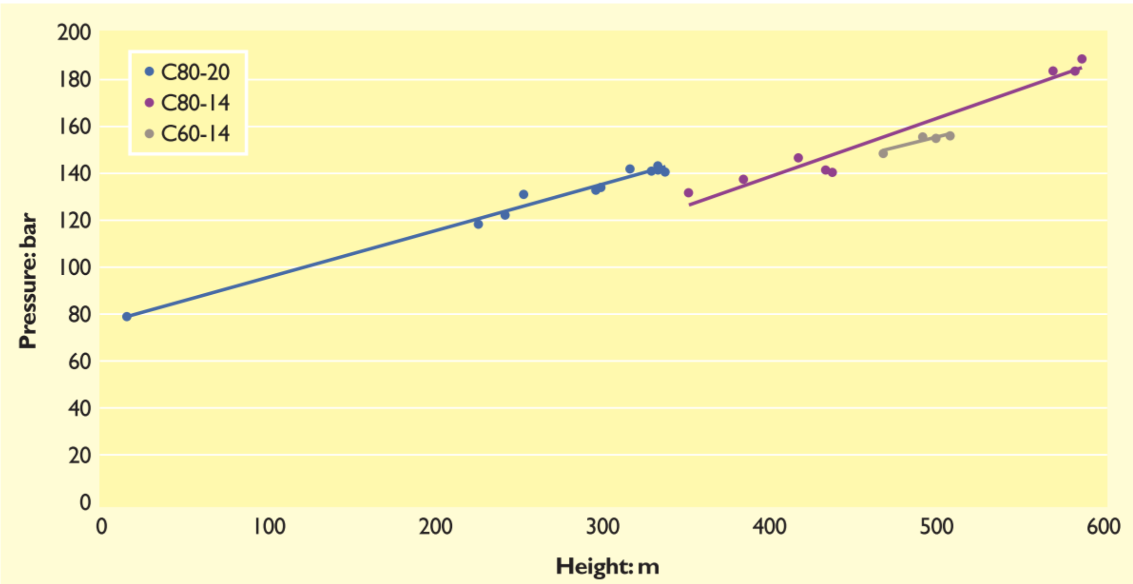


Figure 3.7. Increasing pumping pressure as the construction height of the Burj Khalifa rises (Aldred, 2010).

Supplying sufficiently workable concrete to the rising heights of modern skyscrapers efficiently undoubtedly continues to be challenging but is crucial for the continued use of HSC in super high-rise buildings. But with the development of more powerful pumps in combination with the continuous advances in chemical admixture, even possibly establishing concrete mixing stations mid-height of high-rise buildings to reduce the pumping height, it is certainly possible to find a way to pump high-strength concrete to remarkable heights (Nehdi 2013). This was proved with the success story of pumping the HSC for the Burj Khalifa which took place in one of the most demanding of concreting environments.

Another area where the use of high-strength concrete, or high-performance concrete, is essential is at the construction of structures exposed to aggressive and severely damaging environments. These harmful environments may be encountered in construction projects such as marine and off-shore structures, power plants (mainly hydro and nuclear), sewage networks and generally demanding infrastructure projects such as bridges and crossovers. (Konkov, 2013)

3.2.2 Areas where the use of high-strength concrete can be profitable

While the use of high-strength concrete is almost inevitable when constructing concrete high-rise buildings and off-shore structures, there are also other aspects and application areas which benefit from the use of high-strength concrete. Firstly, the reduced dimensions of load bearing members obtained by using HSC in high-rise buildings is naturally also applicable in more moderately loaded structures. As Yousry and Shikh (2008) demonstrated, by using HSC of various strength classes compared to a normal-strength concrete with 35 MPa compressive strength for columns in 12 story high building, the total costs of the columns could be reduced significantly. The fact that the columns in that study, located in the centre, edge and corners of the building, and subjected to loads in the range of 3-6 MN showed decreasing costs when higher strength concretes were utilized, indicates that the advantages of using HSC can also noticeable in more common buildings where the loads present are relatively small.

Secondly, high-strength concrete can also be applied for making structures and buildings more environmentally sustainable. As cement production is a highly energy consuming process, the cement content of concrete should be minimized to make the material more sustainable. Although HSC usually contain more cement, or at least cementitious materials, per cubic meter than regular normal-strength concrete, the reduced total volume of concrete needed when using HSC can counteract and even outweigh the higher cement content per cubic meter. To highlight this an interesting study conducted by Pons and de la Fuente (2013) investigated how different strength concrete affects the overall sustainability of the resulting reinforced concrete columns in a medium sized residential building. Even for as small loads as these, an axial compression load of 2158 kN and a bending moment of 50 kNm, it was concluded that using higher strength concrete results in more sustainable columns. The most sustainable column alternative in the study proved to be a circular cross-section, compared to a square-shaped, made with 75 MPa self-consolidating concrete, when both SCC and vibrated concrete for the strength classes 25 MPa, 50 MPa and 75 MPa were compared (Pons 2013). Similar results were reached in a study conducted by Park et al. (2013) where steel reinforced concrete columns of a 35-floor building were optimized considering both column costs and CO_2 emissions. In this case it was shown that increasing the use of high-strength materials, both concrete and steel, resulted decreased overall costs and CO_2 emissions although unit costs and emissions for the high-strength materials are relatively high (Park et al).

Furthermore, HSC often contains relatively high portions of supplementary cementitious materials which are industrial by-products, such as fly ash and silica fume, that would otherwise be wasted as landfill (Wu et al., 2010). By using HSC, it is also more realistic to make structures with a longer service life. This aspect should also be considered as a major improvement for a more sustainable construction industry.

Additionally, the use of HSC in concrete-steel composite structures also has it clear advantages compared to NSC. Composite structures combining concrete and steel is a relatively new form of structural elements that maximizes both materials advantages. For columns, these composite members are primarily either in-filled columns, where a hollow steel section is filled with concrete, or encased columns where the concrete encases one or several steel sections, examples of these types are shown in Figure 3.8.

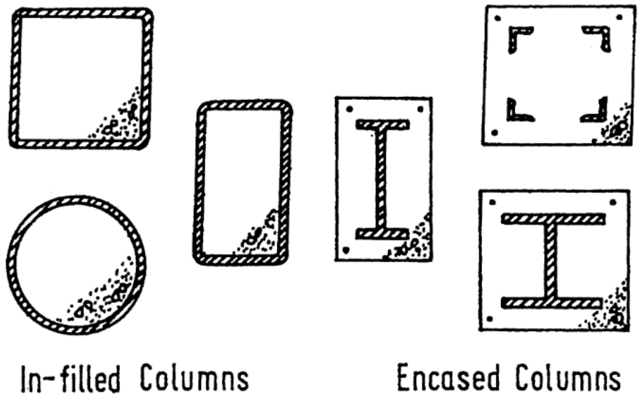


Figure 3.8. Examples of composite columns (Shanmugam and Lakshmi, 2001).

While composite columns made with NSC are an improvement to plain concrete or steel columns, the use of HSC instead can be assumed to improve the composite column even further (Shanmugam and Lakshmi, 2001). This improvement is mainly due to the increasing stiffness, or modulus of elasticity, along with the compressive strength of concrete. For composite encased columns the use of the denser and more impregnable HSC gives the structural steel inside the column a better protection against penetration of harmful substances.

As high-strength concrete still cracks at relatively low tensile stresses, the benefits of high compressive strength of the material is not so obvious in flexural members as in compression members (Neville and Aïtcin 1998). Bridges are good examples of this. Whereas the columns or equivalent structural members carrying the vertical loads of the structure clearly benefits from a material with a higher compressive strength in the same way as columns in normal buildings, the bridge deck does not necessarily benefit to the same extent from a higher compressive strength concrete. However, as the bridge deck must be highly durable and be able to resist the ingress of harmful substances, a low w/cm ratio is undoubtedly beneficial, and with this naturally follows a high compressive strength. Additionally, concrete bridge girders, or concrete beams in general, benefit from the high compressive strength of the concrete by reducing the number of necessary girders, allowing shallower cross-sections and thus lighter structures. The benefits of using HSC in girders or beams is fully exploited when combined with prestressing which, when correctly designed, effectively reduces the cracking of the concrete (Caldarone et al., 2005). The earlier reaching of demoulding strengths connected with the use of HSC naturally also applies to bridge construction and general production of traditional or prestressed beams, whether cast in-situ or precast, and consequently reducing the overall construction time.

Although this thesis focuses on cast in-situ concrete, the use of HSC in the precast industry is also worth mentioning. In factory conditions the casting and curing processes are easier to control and so HSC, and even UHSC or FRHPC, can be used more effectively to create visually stunning concrete elements. These elements with complicated geometries and thin structures that at first glance does not appear to be made of concrete are becoming widely used on facades and making otherwise boring surfaces more aesthetically appealing. Examples of such spectacular precast elements are shown in Figure 3.9, both the façade and balcony elements are made by fibre reinforced high-strength concrete. Naturally also more conventional structural precast elements, such as columns, beams and girders, for various applications are made using higher-than-normal strength concrete when projects benefit from using precast elements or when concrete cast in-situ is not otherwise possible.

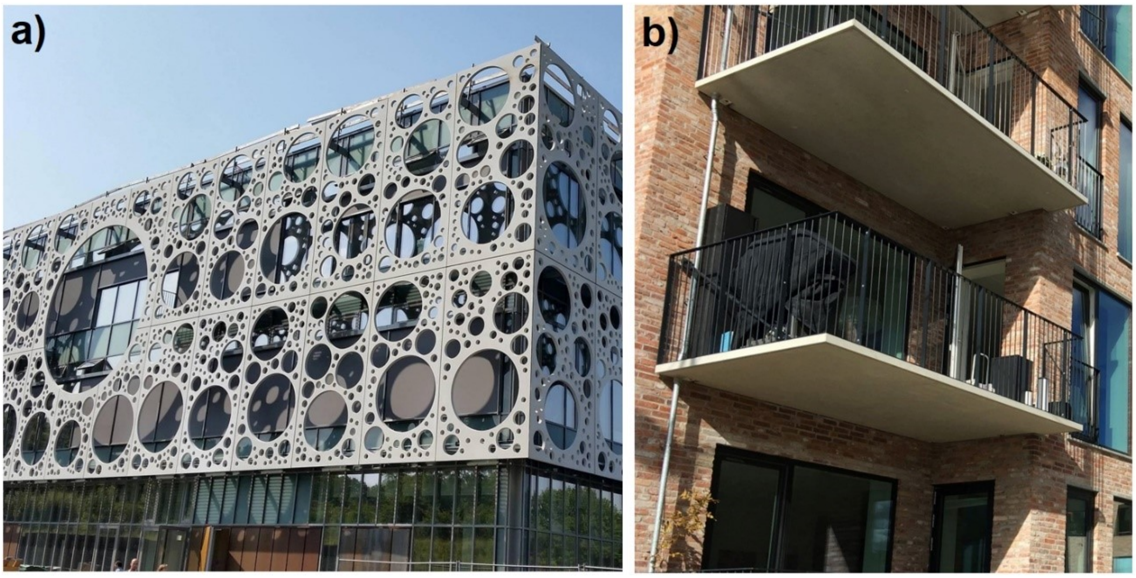


Figure 3.9. a) The façade of The University of Southern Denmark consisting of precast high-strength steel fibre reinforced concrete elements (2014). b) Minimalistic high-strength steel fibre reinforced precast element balconies at the Castor building project in Copenhagen (2018). (hi-con, 2019)

3.2.3 Optimizing column design using high-strength concrete

Since high-strength concrete is mainly used for columns, this thesis will focus on how the column design oh HSC can be optimized and what aspects should be given extra attention. When following the Eurocodes in the design of concrete columns, there are several important aspects determining the controlling factors concerning profitability of the use of high-strength concrete in a column. Firstly, determining the required minimum longitudinal reinforcement, $A_{s,min}$, for a column is governed both by the dimension of the column cross-section as well as the load, as seen in Equation 16 below;

$$A_{s,min} = MAX \begin{cases} \frac{0.10 \times N_{Ed}}{f_{yd}} \\ 0.002 \times A_c \end{cases} \tag{16}$$

where N_{Ed} is the design axial force, f_{yd} is the reinforcements design yield strength and A_c is the gross area of the concrete section.

Therefore, when the loading factor is governing, the minimum required reinforcement steel will be the same for a normal-strength concrete column and an HSC column, even though the HSC could be made with a smaller cross-section. However, the minimum reinforcement for columns is very rarely sufficient since this would require a practically purely compressed column without any considerable bending moment. Since the Eurocodes requires a minimum eccentricity of compression forces according to Equation 17, the presence of a bending moment is inevitable in column design. This bending moment can become significant, especially when the forces and cross-sections are getting great.

$$e_0 = MAX\begin{cases} 20\ mm \\ h/30 \end{cases} \tag{17}$$

where h is the depth of the column.

The brittle nature of HSC compared to NSC is another significant aspect that needs to be considered when designing HSC columns. Firstly, it is important to recognize that the stress-strain curve changes with the increasing compressive strength of concrete from a parabolic-rectangular to an almost triangular shape, as illustrated in Figure 3.10.

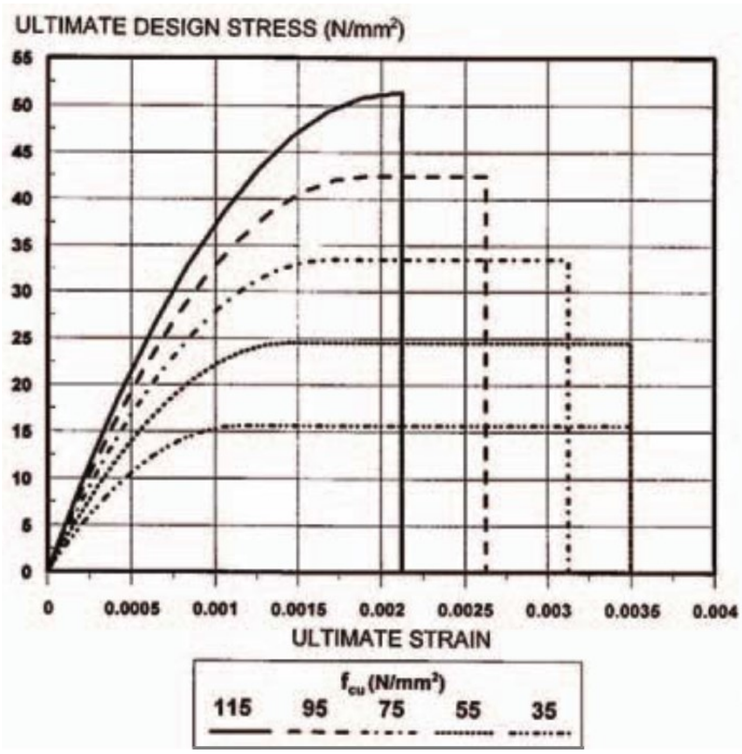


Figure 3.10. Proposed stress blocks for varying concrete strength classes, in cube strengths, according to the BS 8110, highlighting the almost triangular shape connected to high-strength concrete explaining its more brittle failure model (Mcfarlane, 2007).

This is essentially the result of the decreasing ultimate compressive strain of concrete as the compressive strength rises. According to the Eurocode this ultimate compressive strain, ε_{cu2}, can be considered having a constant value of 3,5 ‰ for NSC, $f_{ck} \leq 50$ MPa, while for HSC it decreases with the compressive strength, f_{ck}, according to Equation 18 below;

$$\varepsilon_{cu2}(‰) = 2.6 + 35\,[(90 - f_{ck})/100]^4 \tag{18}$$

Ensuring a sufficiently ductile design of columns made with HSC is thus vital. According to McFarlane (2007), this is achieved most effectively by sufficient core confinement. The transverse reinforcement of columns that provide the core confinement should enable the column to withstand the applied load even after the spalling of the concrete cover shell and thus avoid sudden column failures. Although the design of transverse links is also an important aspect for NSC columns, the detailing for NSC columns regarding transverse reinforcement is not adequate in providing sufficient core confinement and restraint to the longitudinal reinforcement in HSC columns. Thus, when designing HSC columns careful attention should be put on the transverse reinforcement links in terms of spacing, anchoring and general strength of the steel (McFarlane 2007). An important and easy step on the way of reaching an acceptable core confinement for HSC columns are assuring that the ends of the transverse links are bent into the core of the columns and thus anchored properly, demonstrated in Figure 3.11. This applies to both circular and rectangular columns and, as mentioned in Chapter 3.1.5, is also an effective way of increasing the capacity of HSC columns when exposed to fire.

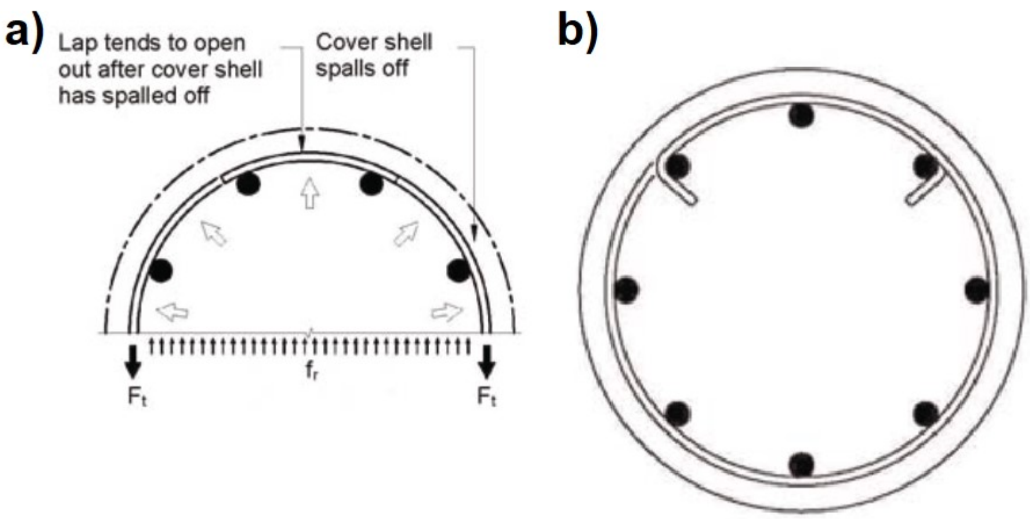


Figure 3.11. a) Illustration of the mechanism of the spalling of the concrete cover and core confinement by inadequate transverse reinforcement links and in b) properly anchored transverse links to provide sufficient core confinement for reinforced HSC columns (McFarlane, 2007).

As the higher strength of HSC will not only enable columns with smaller cross-sections due to its superior resistance to compression forces compared to NSC, but also reduce the need of reinforcement steel because of its contribution in resisting bending moments. The use of HSC thus enables two main optimization options; reducing the dimensions of the columns cross-section or maintaining the cross-section dimensions but reducing the reinforcement steel. Both options have their advantages, economical, esthetical as well as constructability wise. To try and answer whether there exists an optimal balance between these aspects, a hypothetical but realistic, scenario will be evaluated in the following chapter.

4 Column comparison

To understand what effect the choice of concrete class can have on highly loaded reinforced concrete columns, the following imaginary, but realistic, scenario was studied. Simply supported columns of a high-rise building that are subject to purely compressive loads, ranging from 20 MN to 50 MN, were considered for this study. Two different reinforcement approaches were considered; minimum reinforcement and a reinforcement to minimize the cross-section area of the column. For each load case and reinforcement approach, the column was calculated and designed using one normal-strength concrete, C35/45, and three different high-strength concretes, C55/67, C70/85 and C90/105. This evaluation provides an estimate of what savings are realistically achievable by using high-strength concrete. Both from a purely economic aspect when only materials and production costs are evaluated, but also when the increased free floor space of the HSC columns is taken into consideration.

4.1 Structural analysis and considerations

The columns to be evaluated were to represent typical load bearing members in potential future high-rise building projects in Finland. Therefore, loads that can be considered as high, or higher than normally occurring column loads in Finland today, were emphasized. In accordance to this, the varying load cases were chosen as being relevant loads for columns positioned in different areas of the building. For every load case, the column was designed with each of the compared concrete classes as well as utilizing a minimum reinforcement and minimum column cross-section area approach. All calculations relevant in determining the required dimensions and quantities of the column and reinforcements were done in accordance to the Eurocodes.

For the minimum reinforcement option different dimensions of main longitudinal reinforcement bars for different load cases were used to realistically reach the required area of reinforcement, according to Equation 16. For the option of minimizing the area of the column a relatively high reinforcement ratio was utilized, but still under the allowed maximum reinforcement. Additionally, the maximum reinforcement was restricted to what can reasonably be reached by using 32 mm diameter rebars. For this evaluation, and for the sake of comparability, the main longitudinal reinforcements were kept constant for all columns made of different concrete within the same load case. This was expected to clearly illustrate what effects using high-strength concrete can have on the dimensions of columns and simultaneously it enabled the investigation of the economic effects it can have.

The columns were presumed to have a height of 4 meters, the assumed column type and buckling mode for the calculations is shown in Figure 4.1a along with an example drawing of a typical column section, illustrating the relevant column dimensions and placement of reinforcements, Figure 4.1b.

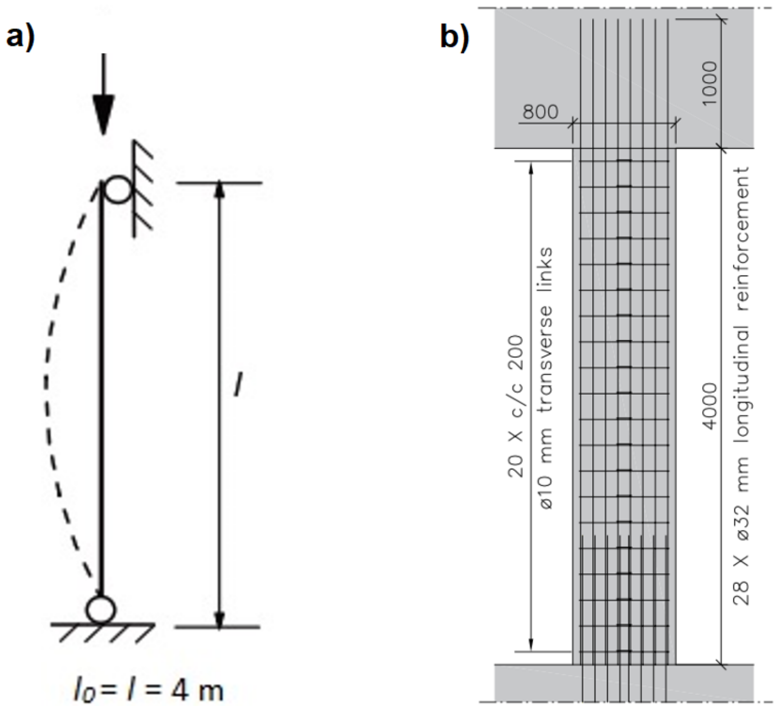


Figure 4.1. a) Assumed column type and buckling type. b) Example of a typical column, 30 MN load case with C70/85 concrete and minimum column area option, and its section view showing the general dimensions of the column and reinforcement principle applied to all column cases.

The transverse reinforcement was not regarded as a notable design parameter in this case and therefore kept constantly as closed loops made of 10 mm rebars with a 200 mm spacing for all the columns. This seemed like a reasonable assumption and was sufficient in all the cases according to the Eurocode regulations concerning transverse reinforcements. However, due to the large cross-sections of the columns, some additional transverse tie-reinforcements in form of cross-ties had to be applied. These cross ties were placed so that no longitudinal reinforcement bar would be further than 150 mm from a bar restrained by a cross tie or by the corner of the transverse tie loop reinforcement, as is required in the Eurocodes. An example of the column reinforcement from the 30 MN load case of both reinforcement options with the concrete class C70/85 is shown in Figure 4.2. The same reinforcement principles are followed for all different load cases and column configurations.

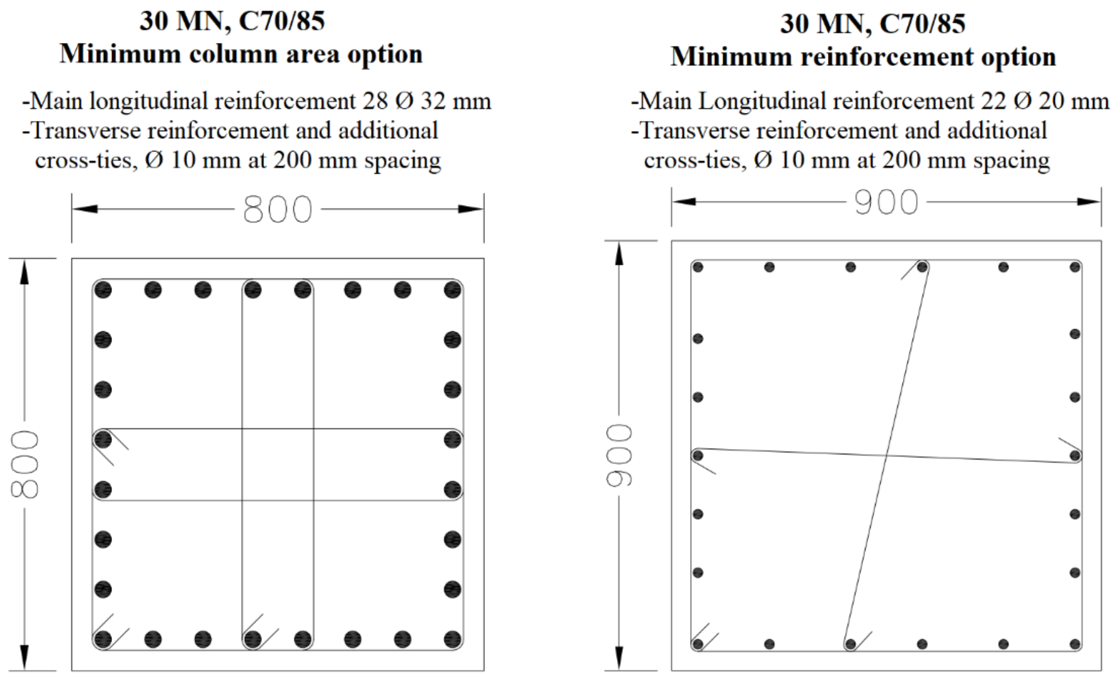


Figure 4.2. Example cross-section views of the reinforcement principles applied for the minimum column area, left, and the minimum reinforcement option, right. Both examples are from the 30 MN load case with class C70/85 concrete.

As the columns were assumed to be located indoors, the exposure class XC1 was chosen for the calculations. Thus, no significant requirements due to environmentally connected exposure classes had to be considered. Similarly, the fire safety aspects were assumed not to affect the structural design. If the chosen concrete cover of 35 mm would not suffice in ensuring the relevant fire safety class, external fireproofing similar for all columns can be added. Other relevant input data for calculating and ensuring that the requirements of the columns are fulfilled according to the Eurocodes were chosen as follows:

- Maximum aggregate size of 16 mm
- Accepted deviation, $\Delta c_{dev} = 10$ mm
- Reinforcement steel strength, $f_{yk} = 500$ MPa
- Working life of 100 years
- Structure class = 1 (rakenneluokka 1), resulting in the following reduced safety factors:
  - The partial safety factor for concrete, $\gamma_C = 1.35$
  - The partial safety factor for reinforcing steel, $\gamma_S = 1.1$
- Coefficient for unfavourable effects on the concrete strength, $\alpha_{CC} = 0.85$
- The only considerable bending moment applied to the columns was that due to the minimum eccentricity of the compression force
  - The minimum eccentricity, $e_0 = MAX\begin{cases} \frac{h}{30} \\ 20\ mm \end{cases}$
  - The determining bending moment, M_{Ed}, thus being $M_{Ed} = e_0 \cdot N_{Ed}$, where N_{Ed} is the applied compression force

Generally, the resistance of a column would not be fully utilized, since some safety is wanted against possible changes in the future that could affecting the loads. But since this is only a hypothetical scenario, the columns in this study were designed with a utilization ratio as close to a hundred percent as possible. This was done by increasing the column dimensions by 50 mm at a time until the required resistance was reached. The columns were largely kept square shaped, but occasionally only one side was decreased by 50 mm to optimize the required consumption of concrete. The usual limiting factor demanding larger column dimensions, or increased reinforcement, was not the compression resistance of the column but the resistance to biaxial bending Equation 19. This although there was no actual external bending moment and the second order moment was neglectable.

$$\left(\frac{M_{Edz}}{M_{Rdz}}\right)^a + \left(\frac{M_{Edy}}{M_{Rdy}}\right)^a \leq 1.0 \tag{19}$$

where:

$M_{Edz/y}$ is the design moment, including 2nd order moment, around the respective axis
$M_{Rdz/y}$ is the moment resistance in the respective direction
a is the exponent;
for circular and elliptical cross-sections: a = 2.0
for rectangular cross-sections:

N_{Ed}/N_{Rd}	0.1	0.7	1.0
a =	1.0	1.5	2.0

with linear interpolation for intermediate values.
where:
N_{Ed} is the design value of axial force
$N_{Rd} = A_c f_{cd} + A_s f_{yd}$, the design axial resistance of the section
where:
A_c is the gross area of the concrete section
A_s is the area of longitudinal reinforcement

Even so, this proved to be an acceptable design criterion since the utilization ratio of the columns remained very high. An example of this high utilization ratio and a typical appearance of the interaction diagram is shown in Figure 4.3, in this case for the minimum column area option of the 30 MN load case and 800 x 800 mm^2 sized with class C70/85 concrete column.

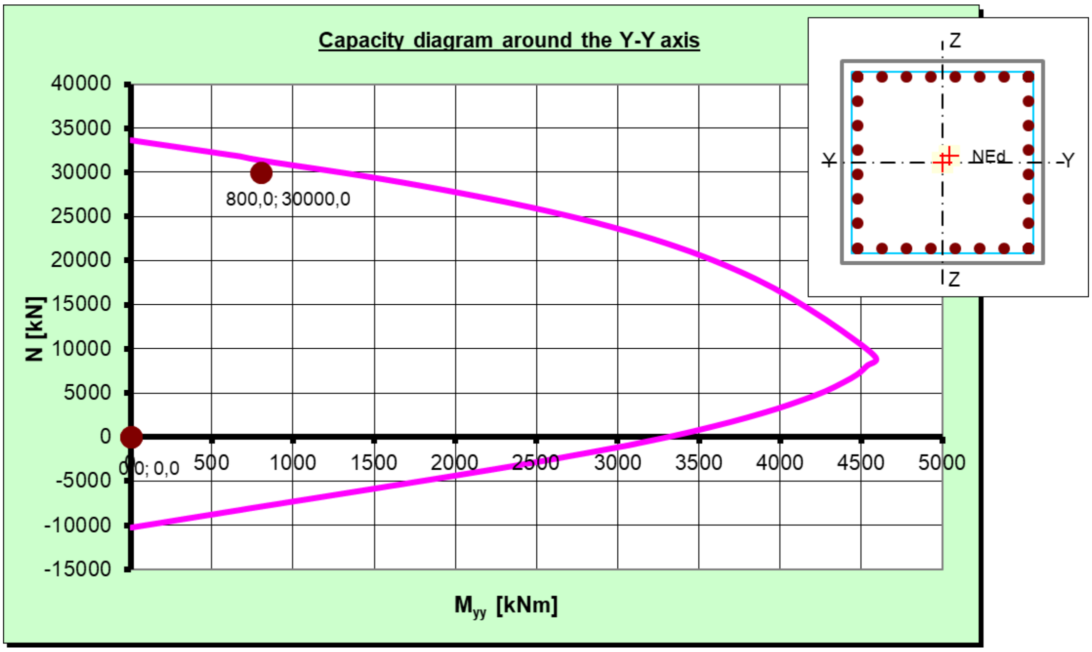


Figure 4.3. Example of a typical interaction diagram from the calculations of the load case 30 MN and the minimum column area option using C70/85 class concrete with resulting 800x800 mm² column cross-section and 28 symmetrically placed 32 mm main rebars.

The resulting column dimensions and specifications from all the load cases can be seen in the Tables 4.1 - 4.4 below. These results will be the base for the cost calculations in the following chapter where the economic aspect of the different column configurations will be evaluated.

Table 4.1. Resulting dimensions and quantities for all column configurations from the 20 MN load case.

20 MN Minimum column area	Profile b [mm]	h [mm]	A_c [m²]	V_c [m³]	Formwork [m²]	$A_{s,\,min}$ [mm²]	Longitudinnal reinforcement, [nTØ]	$A_{s,act}$ [mm²]	Transverse reinforcement* [kg/column]	Total reinforcement [kg/column]
C35/45	850	800	0.68	2.72	13.2	4400	24T32	19302	66.6	824
C55/67	700	650	0.46	1.82	10.8	4400	24T32	19302	55.5	813
C70/85	650	600	0.39	1.56	10.0	4400	24T32	19302	51.8	809
C90/105	600	600	0.36	1.44	9.6	4400	24T32	19302	49.3	807

20 MN Minimum reinforcement	Profile b [mm]	h [mm]	A_c [m²]	V_c [m³]	Formwork [m²]	$A_{s,\,min}$ [mm²]	Longitudinnal reinforcement, [nTØ]	$A_{s,act}$ [mm²]	Transverse reinforcement* [kg/column]	Total reinforcement [kg/column]
C35/45	1000	950	0.95	3.80	15.6	4400	22T16	4423	77.7	251
C55/67	800	800	0.64	2.56	12.8	4400	22T16	4423	64.1	238
C70/85	750	700	0.53	2.10	11.6	4400	22T16	4423	59.2	233
C90/105	700	700	0.49	1.96	11.2	4400	22T16	4423	56.7	230

**Consists of closed loop ties and additional cross ties where necessary, according to the principal shown in Figure 3.1b and 3.2, all with Ø=10 mm and a 200 mm spacing*

Table 4.2. Resulting dimensions and quantities for all column configurations from the 30 MN load case.

30 MN Minimum column area	Profile b [mm]	h [mm]	A_c [m^2]	V_c [m^3]	Formwork [m^2]	$A_{s, min}$ [mm^2]	Longitudinnal reinforcement, [nTØ]	$A_{s,act}$ [mm^2]	Transverse reinforcement* [kg/column]	Total reinforcement [kg/column]
C35/45	1050	1050	1.10	4.41	16.8	6600	28T32	22519	113.4	997
C55/67	900	850	0.77	3.06	14.0	6600	28T32	22519	97.4	981
C70/85	800	800	0.64	2.56	12.8	6600	28T32	22519	88.8	973
C90/105	750	750	0.56	2.25	12.0	6600	28T32	22519	83.8	968

30 MN Minimum reinforcement	Profile b [mm]	h [mm]	A_c [m^2]	V_c [m^3]	Formwork [m^2]	$A_{s, min}$ [mm^2]	Longitudinnal reinforcement, [nTØ]	$A_{s,act}$ [mm^2]	Transverse reinforcement* [kg/column]	Total reinforcement [kg/column]
C35/45	1200	1150	1.38	5.52	18.8	6600	22T20	6912	92.5	364
C55/67	1000	950	0.95	3.80	15.6	6600	22T20	6912	77.7	349
C70/85	900	900	0.81	3.24	14.4	6600	22T20	6912	71.5	343
C90/105	850	850	0.72	2.89	13.6	6600	22T20	6912	67.8	339

**Consists of closed loop ties and additional cross ties where necessary, according to the principal shown in Figure 3.1b and 3.2, all with Ø=10 mm and a 200 mm spacing*

Table 4.3 Resulting dimensions and quantities for all column configurations from the 40 MN load case.

40 MN Minimum column area	Profile b [mm]	h [mm]	A_c [m^2]	V_c [m^3]	Formwork [m^2]	$A_{s, min}$ [mm^2]	Longitudinnal reinforcement, [nTØ]	$A_{s,act}$ [mm^2]	Transverse reinforcement* [kg/column]	Total reinforcement [kg/column]
C35/45	1300	1250	1.63	6.50	20.4	8800	30T32	24127	136.9	1084
C55/67	1050	1000	1.05	4.20	16.4	8800	30T32	24127	112.2	1059
C70/85	950	950	0.90	3.61	15.2	8800	30T32	24127	103.6	1051
C90/105	900	900	0.81	3.24	14.4	8800	30T32	24127	98.6	1046

40 MN Minimum reinforcement	Profile b [mm]	h [mm]	A_c [m^2]	V_c [m^3]	Formwork [m^2]	$A_{s, min}$ [mm^2]	Longitudinnal reinforcement, [nTØ]	$A_{s,act}$ [mm^2]	Transverse reinforcement* [kg/column]	Total reinforcement [kg/column]
C35/45	1400	1350	1.89	7.56	22	8800	18T25	8836	107.3	454
C55/67	1150	1100	1.27	5.06	18	8800	18T25	8836	88.8	436
C70/85	1050	1000	1.05	4.20	16.4	8800	18T25	8836	81.4	428
C90/105	1000	950	0.95	3.80	15.6	8800	18T25	8836	77.7	424

**Consists of closed loop ties and additional cross ties where necessary, according to the principal shown in Figure 3.1b and 3.2, all with Ø=10 mm and a 200 mm spacing*

Table 4.4. Resulting dimensions and quantities for all column configurations from the 50 MN load case.

50 MN Minimum column area	Profile b [mm]	h [mm]	A_c [m^2]	V_c [m^3]	Formwork [m^2]	$A_{s, min}$ [mm^2]	Longitudinnal reinforcement, [nTØ]	$A_{s,act}$ [mm^2]	Transverse reinforcement* [kg/column]	Total reinforcement [kg/column]
C35/45	1450	1400	2.03	8.12	22.8	11000	36T32	28953	151.7	1288
C55/67	1150	1150	1.32	5.29	18.4	11000	36T32	28953	123.3	1260
C70/85	1100	1050	1.16	4.62	17.2	11000	36T32	28953	117.1	1254
C90/105	1000	1000	1.00	4.00	16.0	11000	36T32	28953	108.5	1245

50 MN Minimum reinforcement	Profile b [mm]	h [mm]	A_c [m^2]	V_c [m^3]	Formwork [m^2]	$A_{s, min}$ [mm^2]	Longitudinnal reinforcement, [nTØ]	$A_{s,act}$ [mm^2]	Transverse reinforcement* [kg/column]	Total reinforcement [kg/column]
C35/45	1550	1500	2.33	9.30	24.4	11000	24T25	11781	118.4	581
C55/67	1250	1250	1.56	6.25	20.0	11000	24T25	11781	97.4	560
C70/85	1150	1150	1.32	5.29	18.4	11000	24T25	11781	90.0	552
C90/105	1100	1050	1.16	4.62	17.2	11000	24T25	11781	85.1	547

**Consists of closed loop ties and additional cross ties where necessary, according to the principal shown in Figure 3.1b and 3.2, all with Ø=10 mm and a 200 mm spacing*

4.2 Cost evaluation

For the cost evaluation part of the column comparison, the relevant parts contributing to the total cost of a column were evaluated. Some aspects, such as work site labour costs and concrete pumping, were excluded since they present no noticeable cost difference for the different column types. Another aspect which is difficult to accurately value is the true profits of increased free floor space obtained by minimizing the area occupied by load-bearing members which is made possible by using higher strength concrete. Besides the free floor space gained due to smaller and fewer load bearing members, smaller columns may also be more aesthetically pleasing, which is even harder to put an actual price on. Additionally, smaller columns also contribute in decreasing the total building weight, enabling savings to be made at the foundations.

The more tangible cost aspects contributing to the final expenses of a reinforced concrete column are realistic estimations confirmed by experts in the corresponding field. Firstly, the costs of the different concrete classes had to be established. Since only normal-strength concretes are continually produced and widely available as ready mixed concretes in Finland, only the C35/45 class of the concretes compared in this study was publicly priced. Therefore, it was thought that the concrete prices used in this evaluation should be based on the raw material costs plus a buffer which considers the production and quality control expenses. The raw material costs for concrete applied in this study can be seen in Table 4.5, transportation costs to the concrete factory are included in these prices.

Table 4.5. Assumed prices for the raw materials used in the studied concretes.

	€/tn	€/kg
Cement, Mega (1	100	0.1
Cement, SR (2	110	0.11
Silica fume	300	0.3
Fly ash	25	0.025
Aggregate	20	0.02
Superplasticizer	1000	1

1) Mega-cement by Finnsementti, CEM I 52.5 N, see also Section 4.1.

2) SR-cement by Finnsementti, CEM I 42.5 N - SR, see also Section 4.1.

All the material costs as well as the final concrete prices were confirmed by Rudus Oy (Rudus, 2018a) to be within realistic limits, and more importantly resulting in a sensible price ratio between the concrete classes evaluated in this study. The contents and resulting prices of each concrete class that were used in this evaluation are shown in Table 4.6. However, the fact that the highest strength concrete, the C90/105, is only modified by reducing the water content of the C70/85 class concretes recipe might have resulted in a rather too low price for the C90/105 concrete, which will be discussed more thoroughly later. Since the mix design for the C35/45, C55/67 and C70/85 were provided by Rudus Oy, one aspect of this thesis became to investigate how easily even higher strength concretes could be achieved. Thus, the C90/105 concrete is a highly experimental concrete and would undoubtedly require extensive testing before becoming publicly available. Especially the raw material contents and proportions of this concrete could probably be optimized further. More about the performance of this experimental concrete follows in Chapter 5.

Table 4.6. Compositions of the evaluated concrete classes and the resulting price components of the total cost and final prices including a price margin covering additional production costs.

	C35/45:		C55/67:		C70/85:		C90/105:	
	kg/m^3	€/m^3	kg/m^3	€/m^3	kg/m^3	€/m^3	kg/m^3	€/m^3
Cement	299	29.9	419	41.9	518	57.0	528	58.1
Silica	0	0.0	0	0.0	27	8.1	28	8.4
Fly ash	84	2.1	118	3.0	0	0.0	0	0.0
Aggregate	1812	36.2	1670	33.4	1696	33.9	1727	34.5
SP	2.2	2.2	3.9	3.9	5.1	5.1	9.8	9.8
Water	188	0	187	0	183	0	163	0
Total		70.5		82.2		104.1		110.8
Plus price margin:	*35 %*	**108**	*50 %*	**164**	*50 %*	**208**	*50 %*	**222**

The price buffer, or price margin, was chosen so that the final price of the C35/45 concrete would coincide with the publicly available price of that concrete, minus about 25 %, which is a reasonable discount for contractors. For the high-strength concretes the price buffer was chosen to be slightly larger. This is a fair assumption since HSC is more difficult to produce and requires more attention to quality control than the accustomed normal-strength concretes. Additionally, the introduction of new concrete types may always cause hidden expenses that are difficult to foresee, thus the price buffer must also cover these. However, once the use of these new concretes has become more common, the prices will undoubtedly drop accordingly, even though the quantities of HSC needed will presumably stay comparatively small.

Regarding the transportation costs of concrete, some assumptions were made. It was assumed that full concrete trucks could always be used to transport the ready-mix concrete the hypothetical distance of 8 km to the worksite from the concrete plant. Furthermore, the delivery of the ready-mix concrete would take place during normal working hours with a concrete truck capable of pumping the concrete as well. According to the pricing list of Rudus Oy, this would result in a cost of 15 €/m^3 concrete. For the pumping of the concrete it was assumed, rather optimistically perhaps, that it could be done in half an hour. This would result in a cost of 234.1 €/truck, or 18 €/m^3 concrete, for a truck capable of pumping the concrete over 30 meters and containing 13 m^3 of concrete (Rudus, 2018b). The combined cost of transportation and pumping of concrete thus becomes 33 €/m^3 concrete. Worth mentioning is that in order to fully benefit from the smaller amount of required concrete when using high-strength concrete, a more general scenario must be assumed where at least several columns can be cast at once and consequently not one single concrete truck would be sufficient for the operation. Also, trucks with a capacity of 13 m^3 are admittedly the top-of-the-range equipment, and trucks with capacities of 6-10 m^3 are more commonly used, which would result in even larger benefits when smaller quantities of concrete is required.

The use of prefabricated reinforcement units was the chosen method for installing the column reinforcements in this evaluation. These elements would be transported and lifted into place at the worksite. This system is often preferred to manually install the reinforcement at the worksite both due to economic advantages, but also for minimizing the work done at site. According to information from Finnish reinforcement suppliers, the cost of reinforcements of this type is typically in the range of 1.0-1.1 €/kg steel. This variation depending on the quantity of the ordered series and how densely reinforced the units are. Thus, it was decided that for the minimum reinforcement columns option and for the more densely reinforced minimized column area option, the values of 1.1 €/kg and 1.0 €/kg steel would be used, respectively. Concerning the transportation of the reinforcement elements, a 100 km transportation distance was assumed as well as 20 elements could be transported on one truck. This, in combination with a realistic transportation cost of 3 €/km, resulted in total transportation costs of approximately 15 €/column for the prefabricated reinforcement units. Additionally, a cost of 35 €/column reinforcement was estimated for lifting the elements in place at the worksite.

Due to the relatively large size of the columns, the costs for the formwork was estimated by the area of required formwork for each column, i.e. the area of the column sides. For smaller columns, with cross-sectional dimensions between 200 x 200 mm^2 and 600 x 600 mm^2, the option of standardized column forms would be available at a unit price of 16 €/day according to PERI (2018), a major formworks provider in Finland. However, for larger columns, which would be needed in this evaluation, a realistic cost estimation for the formwork is 1.50 €/m^2 per day (Peri, 2018). Therefore, this value was used for the following calculations. Furthermore, since this cost is per day, to get the total cost of the formworks, it was assumed that the formwork would be needed for 7 days at the construction site for each column.

The earlier calculations regarding the quantities of concrete, reinforcement steel and formwork, Tables 4.1-4.4, were used for calculating the final costs of each column type conducted below. Also, all the cost calculation tables contain the previously explained assumptions for additional expenses, which can be summarized as:

- Transporting the concrete 8 km during normal working hours, using full concrete trucks (13 m^3 concrete/truck) capable of pumping the concrete within half an hour upon arrival at the worksite, 15 + 18 = **33 €/m^3**
- Transportation of prefabricated reinforcement units, **15 €/column**
- Lifting of prefabricated reinforcement units in place, **35 €/column**
- Formwork required at the construction site for the duration of **7 days** per column

The resulting cost calculations and total costs for each column type for the different loading cases are shown in the following Tables 4.7-4.10.

Table 4.7. Cost breakdown for all the column configurations for the 20 MN load case based on the previously calculated quantities.

20 MN Minimum column area	Concrete [€/m^3]	Transport and pumping [€/m^3]	Reinforcement [€/kg]	Formwork [€/m^2·d]	Concrete [€/column]	Transport and pumping [€/column]	Reinforcement [€/column]	Formwork [€/column]	Total cost [€]
C35/45	108	33	1.0	1.5	293.8	89.8	874.2	138.6	**1396**
C55/67	164	33	1.0	1.5	298.5	60.1	863.1	113.4	**1335**
C70/85	208	33	1.0	1.5	324.5	51.5	859.4	105.0	**1340**
C90/105	222	33	1.0	1.5	319.7	47.5	856.9	100.8	**1325**

20 MN Minimum reinforcement	Concrete [€/m^3]	Transport and pumping [€/m^3]	Reinforcement [€/kg]	Formwork [€/m^2·d]	Concrete [€/column]	Transport and pumping [€/column]	Reinforcement [€/column]	Formwork [€/column]	Total cost [€]
C35/45	108	33	1.1	1.5	410.4	125.4	326.4	163.8	**1026**
C55/67	164	33	1.1	1.5	419.8	84.5	311.5	134.4	**950**
C70/85	208	33	1.1	1.5	436.8	69.3	306.1	121.8	**934**
C90/105	222	33	1.1	1.5	435.1	64.7	303.4	117.6	**921**

Table 4.8. Cost breakdown for all the column configurations for the 30 MN load case based on the previously calculated quantities.

30 MN Minimum column area	Concrete [€/m^3]	Transport and pumping [€/m^3]	Reinforcement [€/kg]	Formwork [€/m^2·d]	Concrete [€/column]	Transport and pumping [€/column]	Reinforcement [€/column]	Formwork [€/column]	Total cost [€]
C35/45	108	33	1.0	1.5	476.3	145.5	1047.3	176.4	**1846**
C55/67	164	33	1.0	1.5	501.8	101.0	1031.3	147.0	**1781**
C70/85	208	33	1.0	1.5	532.5	84.5	1022.7	134.4	**1774**
C90/105	222	33	1.0	1.5	499.5	74.3	1017.7	126.0	**1717**

30 MN Minimum reinforcement	Concrete [€/m^3]	Transport and pumping [€/m^3]	Reinforcement [€/kg]	Formwork [€/m^2·d]	Concrete [€/column]	Transport and pumping [€/column]	Reinforcement [€/column]	Formwork [€/column]	Total cost [€]
C35/45	108	33	1.1	1.5	596.2	182.2	450.2	197.4	**1426**
C55/67	164	33	1.1	1.5	623.2	125.4	433.9	163.8	**1346**
C70/85	208	33	1.1	1.5	673.9	106.9	427.1	151.2	**1359**
C90/105	222	33	1.1	1.5	641.6	95.4	423.0	142.8	**1303**

Table 4.9. Cost breakdown for all the column configurations for the 40 MN load case based on the previously calculated quantities.

40 MN Minimum column area	Concrete [€/m^3]	Transport and pumping [€/m^3]	Reinforcement [€/kg]	Formwork [€/m^2·d]	Concrete [€/column]	Transport and pumping [€/column]	Reinforcement [€/column]	Formwork [€/column]	Total cost [€]
C35/45	108	33	1.0	1.5	702.0	214.5	1133.9	214.2	**2265**
C55/67	164	33	1.0	1.5	688.8	138.6	1109.2	172.2	**2109**
C70/85	208	33	1.0	1.5	750.9	119.1	1100.6	159.6	**2130**
C90/105	222	33	1.0	1.5	719.3	106.9	1095.6	151.2	**2073**

40 MN Minimum reinforcement	Concrete [€/m^3]	Transport and pumping [€/m^3]	Reinforcement [€/kg]	Formwork [€/m^2·d]	Concrete [€/column]	Transport and pumping [€/column]	Reinforcement [€/column]	Formwork [€/column]	Total cost [€]
C35/45	108	33	1.1	1.5	816.5	249.5	549.5	231.0	**1846**
C55/67	164	33	1.1	1.5	829.8	167.0	529.2	189.0	**1715**
C70/85	208	33	1.1	1.5	873.6	138.6	521.0	172.2	**1705**
C90/105	222	33	1.1	1.5	843.6	125.4	516.9	163.8	**1650**

Table 4.10. Cost breakdown for all the column configurations for the 50 MN load case based on the previously calculated quantities.

50 MN Minimum column area	Concrete [€/m^3]	Transport and pumping [€/m^3]	Reinforcement [€/kg]	Formwork [€/m^2·d]	Concrete [€/column]	Transport and pumping [€/column]	Reinforcement [€/column]	Formwork [€/column]	Total cost [€]
C35/45	108	15	1.0	1.5	877.0	121.8	1338.1	239.4	**2576**
C55/67	164	15	1.0	1.5	867.6	79.4	1309.7	193.2	**2450**
C70/85	208	15	1.0	1.5	961.0	69.3	1303.5	180.6	**2514**
C90/105	222	15	1.0	1.5	888.0	60.0	1294.9	168.0	**2411**

50 MN Minimum reinforcement	Concrete [€/m^3]	Transport and pumping [€/m^3]	Reinforcement [€/kg]	Formwork [€/m^2·d]	Concrete [€/column]	Transport and pumping [€/column]	Reinforcement [€/column]	Formwork [€/column]	Total cost [€]
C35/45	108	15	1.1	1.5	1004.4	139.5	688.9	256.2	**2089**
C55/67	164	15	1.1	1.5	1025.0	93.8	665.8	210.0	**1995**
C70/85	208	15	1.1	1.5	1100.3	79.4	657.7	193.2	**2031**
C90/105	222	15	1.1	1.5	1025.6	69.3	652.2	180.6	**1928**

As can be seen from the tables above, and more clearly from the Table 4.11 below, summarizing the final costs of the various combinations evaluated, the use of higher strength concrete in columns does not seemingly appear to provide any significant economic benefits.

Table 4.11. Summary of the resulting total costs for each column configuration.

	Column loads			
	20 MN	30 MN	40 MN	50 MN
Minimum column area option	**Total column costs for each load case [€]**			
C35/45	1396	1846	2265	2576
C55/67	1335	1781	2109	2450
C70/85	1340	1774	2130	2514
C90/105	1325	1717	2073	2411

	20 MN	30 MN	40 MN	50 MN
Minimum reinforcement option	**Total column costs for each load case [€]**			
C35/45	1026	1426	1846	2089
C55/67	950	1346	1715	1995
C70/85	934	1359	1705	2031
C90/105	921	1303	1650	1928

However, many of the advantages of using HSC has not been taken into consideration at this stage of the evaluation due to the difficulties of accurately valuating these advantages. In the following chapter a more thorough evaluation of the results will be conducted.

4.3 Discussion of the column comparison results

Even though this case study indicates that the purely economic benefits of using high-strength concrete in columns are not very significant, many advantages of using HSC were left unconsidered. One of the main advantages of using HSC left unconsidered was the space saved. As can be seen from Figure 4.4, the cross-section area of all the HSC columns are considerably smaller than the compared NSC column for all load cases.

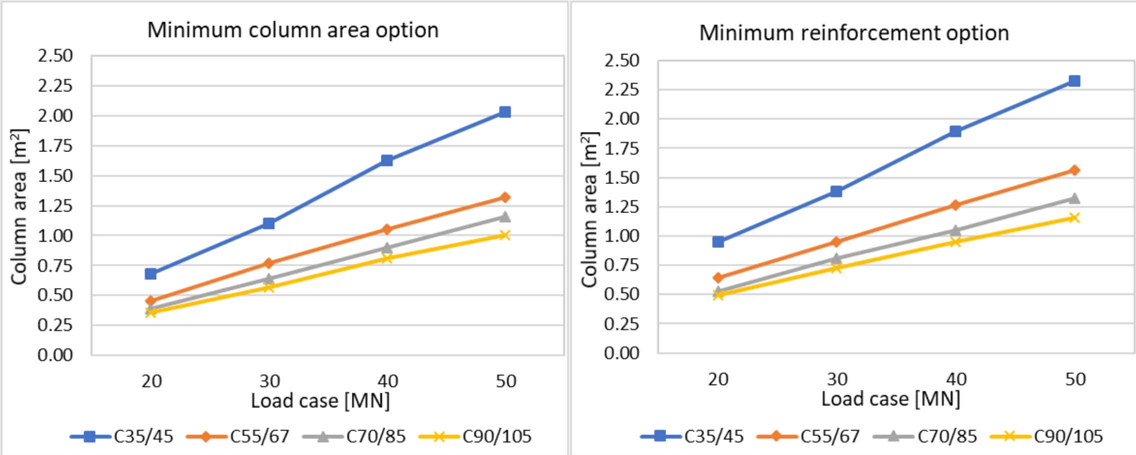


Figure 4.4. Graph illustrating the obtainable reductions in single column areas by using higher strength concretes.

The reduction of area per column compared to the C35/45 concrete column ranges from 31 % to 35 %, 41 % to 45 % and 47 % to 51 % for the C55/67, C70/85 and C90/105 columns, respectively. In addition to the decreased absolute area of one column, the smaller required cross-section area of HSC columns gives the designer more freedom for optimizing the final free floor area as well as adjusting the cross-sections of the columns throughout the building. The higher loadbearing capacity of HSC columns with equal dimensions as regular concrete columns could for instance be utilized to reduce the total number of columns needed, and thus liberate floor space in a more optimal way than by simply reducing the cross-section area of the columns. Using different strength classes of concrete for columns on different levels of the building also gives the opportunity to maintain the same column dimensions throughout the whole building, thus facilitating the reuse of the same column formworks throughout parts, or all of the floors of the building.

From Figure 4.4 it can also be observed that the variation in area between the three high-strength concrete columns is relatively small. This can be explained by that as the cross-section dimensions decreases, the moment arm of the columns resisting moment also decreases. Thus, as the reinforcement in this case study was kept constant for the different column types, the resisting moment of the column also decreases even though the concrete compressive strength increases. Although no significant bending moment was applied in this case study, the effect of the biaxial bending was generally the limiting factor for decreasing the cross-section dimensions of the columns further. The moment resistance of the columns could of course be increased by adding more reinforcing steel, but since steel is relatively expensive this would be an economically unattractive option. Therefore, it is safe to say that focusing simply on decreasing the dimensions of the columns by increasing the reinforcement ratio is not an appropriate design approach, especially when significant external moments must be taken into consideration.

A clearer overview of how the use of different concrete strength classes influence the total costs per column can be found from Figure 4.5. From these graphs it can be confirmed that as the loading increases the discrepancies between the final costs also grow. This indicates that especially for greater load cases, the choice of concrete strength class can have a significant effect on the final costs of the columns. However, admittedly the concrete C90/105 was only a simply modified version of the C70/85 concrete, with less mixing water to reach a lower w/cm ratio. Therefore, as this mix design was used for the cost calculation for the concrete price, the price for such a high strength concrete as class C90/105 was perhaps somewhat optimistically low. Consequently, the final costs presented in this study concerning all columns made with the C90/105 concrete are also slightly optimistic. Even so, a pattern that less expensive columns can be obtained by using HSC is clearly visible.

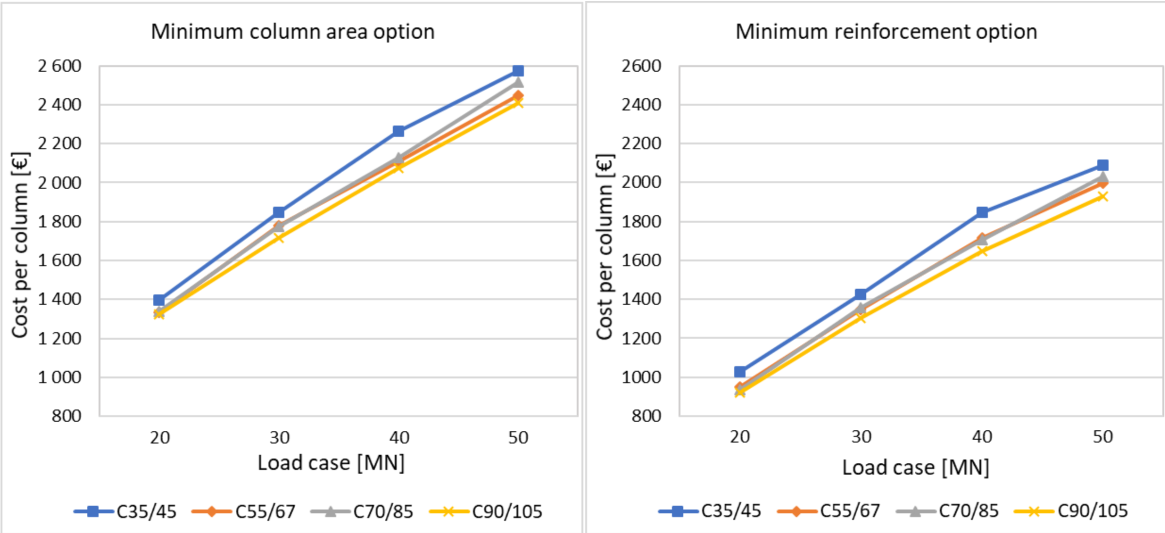


Figure 4.5. Overview of the varying column costs for the different load cases and depending on used concrete strength class when only material and production costs are evaluated.

As this study only focused on two column options, minimizing the reinforcement steel and minimizing the column dimensions, the seemingly most economically viable option would be the columns applying the minimum reinforcement principal. However, as mentioned earlier, the increased free floor space due to reduced column sizes was not accounted for in any way at this point of the evaluation. Thus, the final cost of a column is heavily influenced by how valuable the floor space in the building is. Yet, it is unlikely that the minimizing the column area to extreme extents would be a very economic option since this requires adding large quantities of reinforcement steel or using concretes with a very high compressive strength. This aspect is of course greatly influenced by the price of steel. From Figure 4.6, where the portions making up the total cost of a column are presented, it can be noted that the cost portion of the reinforcement steel is greatly increased for the minimized column area option.

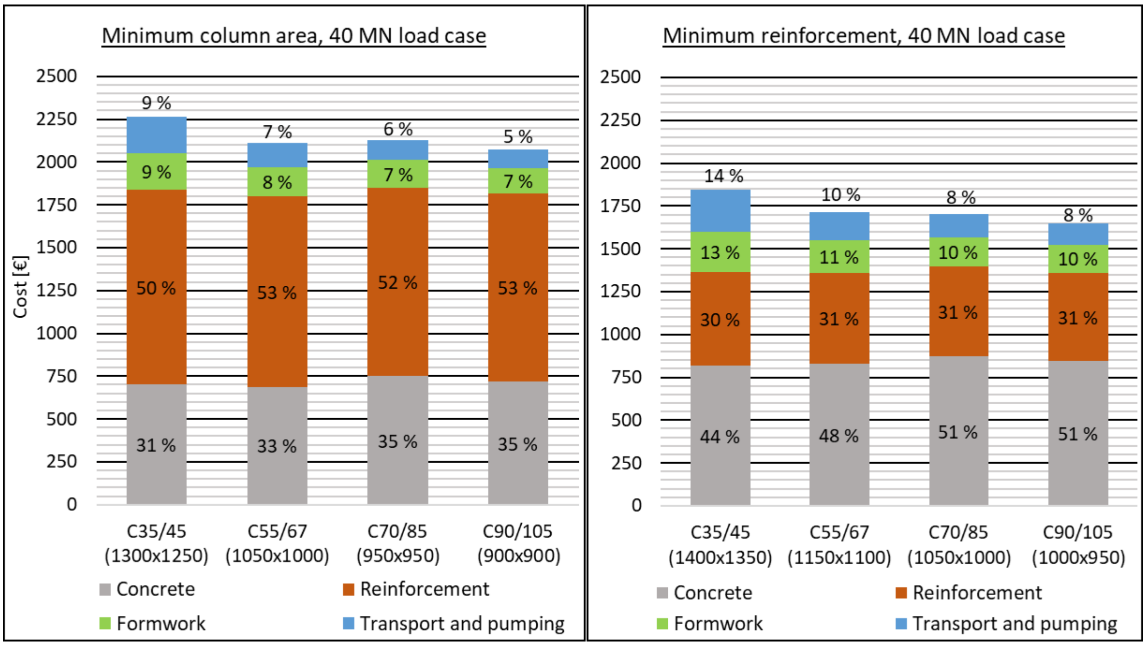


Figure 4.6. Cost portions of the total costs for the different column types for the 40 MN load case. The resulting dimensions (mm) of the columns in parenthesis under the used concrete strength class.

As can be observed from Figure 4.6, the proportions of the cost portions remained very similar although the concrete classes and column dimensions changed. Naturally the cost portions for formwork and transportation and pumping decreased as the dimension of the columns could be reduced with rising concrete strength classes. As the quantities of reinforcement were kept constant for all four concrete classes in each column type, the only change in the total steel consumption per column origins from the varying tie lengths as the column dimensions changes. Thus, the absolute costs of the reinforcing decreased slightly with the rising strength class. But since the overall column costs also decreased, the cost portion of the reinforcements rose slightly in proportion to the total cost of a column. The absolute cost of concrete remained similar for all column types although the assumed prices and quantities of concrete differed quite significantly. This indicates that opting for an inexpensive regular concrete is not more economical in the end, especially when considering the additional gains high-strength concrete offers. Similarly, the minimum reinforced column option is undoubtedly not always the most economic option. The optimum reinforcement ratio is always going to be case specific, where the need of free space, low costs and future structural flexibility must be compared and evaluated against each other.

However, even without considering the advantages of increased free floor space and improved durability which high-strength concrete offers, columns made of HSC seem to be slightly more financially viable than the normal-strength concrete columns. Furthermore, it can be assumed that once the higher strength concrete classes become more widely used and produced, the prices of these concrete grades will drop. It should also be noted that the transport distance and pumping times were kept to a minimum in this study. Thus, when the distances and pumping times inevitably increases, the costs from these factors become more significant and the advantages of the smaller quantities of concrete needed for HSC columns will truly start showing.

Although it is challenging to accurately assess the value of increased free floor space connected to the use of high-strength concrete in columns, an attempt to do so was made in order to give a more realistic view of what the overall economic benefits of using HSC are. By estimating the free floor value for the studied hypothetical building to be either 2000 €/m^2 or 5000 €/m^2, two cases representing bottom and top floor valuation cases, gives a realistic range of savings obtainable by using HSC depending on how valuable the free floor space in the relevant project is. To obtain the final costs of each column configuration, the areas of each column case were compared to the smallest column area of that column case, i.e. the column made with the highest concrete strength class, C90/105. The difference in column areas were then multiplied by the mentioned floor values and added to the previously presented column costs in Table 4.11. The additional floor area required by columns made with lower strength concrete and the corresponding additional costs are presented in Table 4.12. The resulting final cost for each column configuration is presented in Table 4.13 and plotted against each other in Figure 4.7.

Table 4.12. Differences in column areas compared to columns made with C90/105 concrete and the resulting additional costs from the increased column areas when floor area was valued at 2000 €/m^2 and 5000 €/m^2.

	20 MN			30 MN			40 MN			50 MN		
Minimum column area option	Differences in column areas and cost additions when floor area valued at;			Differences in column areas and cost additions when floor area valued at;			Differences in column areas and cost additions when floor area valued at;			Differences in column areas and cost additions when floor area valued at;		
		2000 €/m^2	5000 €/m^2		2000 €/m^2	5000 €/m^2		2000 €/m^2	5000 €/m^2		2000 €/m^2	5000 €/m^2
	[m^2]	[€]	[€]	[m^2]	[€]	[€]	[m^2]	[€]	[€]	[m^2]	[€]	[€]
C35/45	0.32	640	1600	0.54	1080	2700	0.82	1630	4075	1.03	2060	5150
C55/67	0.10	190	475	0.20	405	1013	0.24	480	1200	0.32	645	1613
C70/85	0.03	60	150	0.08	155	388	0.09	185	463	0.16	310	775
C90/105	0.00	0	0	0.00	0	0	0.00	0	0	0.00	0	0
Minimum reinforcement option	[m^2]	2000 €/m^2	5000 €/m^2	[m^2]	2000 €/m^2	5000 €/m^2	[m^2]	2000 €/m^2	5000 €/m^2	[m^2]	2000 €/m^2	5000 €/m^2
C35/45	0.46	920	2300	0.66	1315	3288	0.94	1880	4700	1.17	2340	5850
C55/67	0.15	300	750	0.23	455	1138	0.32	630	1575	0.41	815	2038
C70/85	0.04	70	175	0.09	175	438	0.10	200	500	0.17	335	838
C90/105	0.00	0	0	0.00	0	0	0.00	0	0	0.00	0	0

Table 4.13. Final costs for all column configurations when additional costs for larger column areas is added to the total column costs from material and production expenses.

	Load cases				Load cases			
	20 MN	30 MN	40 MN	50 MN	20 MN	30 MN	40 MN	50 MN
Minimum column area option	*Final cost for each column configuration when 2000 €/m^2 is added to the total costs*				*Final cost for each column configuration when 5000 €/m^2 is added to the total costs*			
C35/45	2036	2926	3895	4636	2996	4546	6340	7726
C55/67	1525	2186	2589	3095	1810	2794	3309	4062
C70/85	1400	1929	2315	2824	1490	2162	2593	3289
C90/105	1325	1717	2073	2411	1325	1717	2073	2411

	20 MN	30 MN	40 MN	50 MN	20 MN	30 MN	40 MN	50 MN
reinforcement option	*Final cost for each column configuration when 2000 €/m^2 is added to the total costs*				*Final cost for each column configuration when 5000 €/m^2 is added to the total costs*			
C35/45	1946	2741	3726	4429	3326	4713	6546	7939
C55/67	1250	1801	2345	2810	1700	2484	3290	4032
C70/85	1004	1534	1905	2366	1109	1797	2205	2868
C90/105	921	1303	1650	1928	921	1303	1650	1928

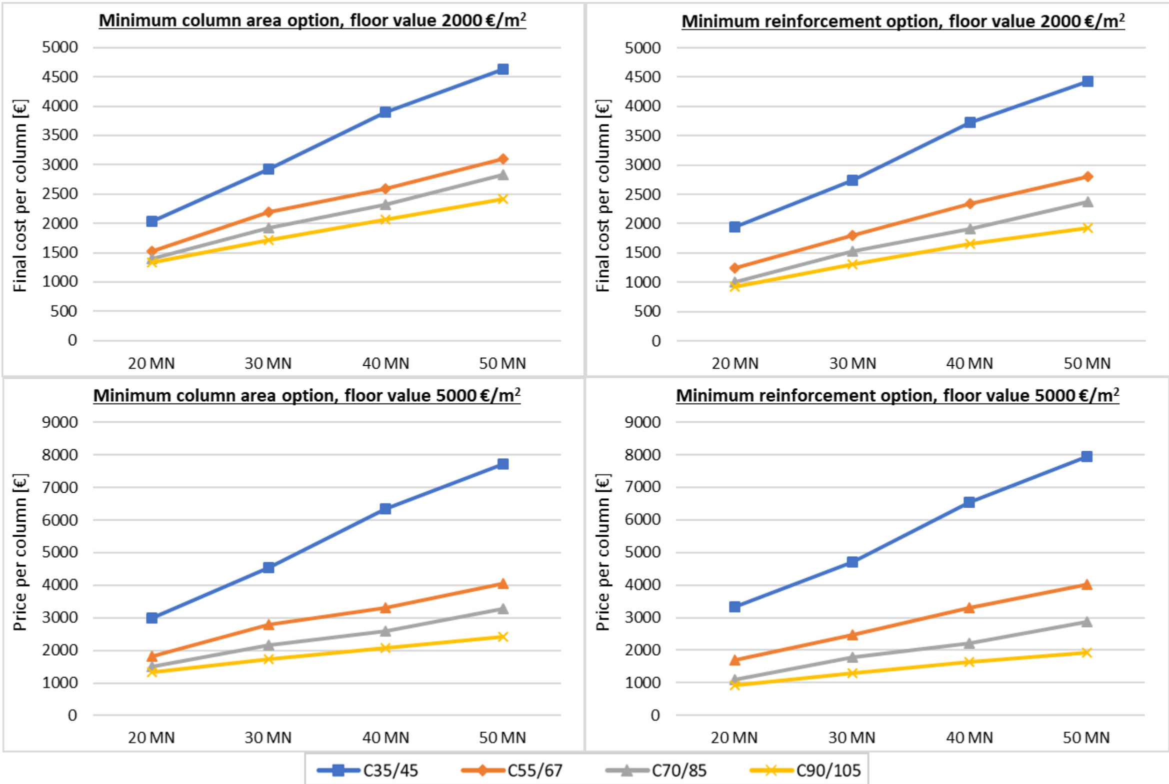


Figure 4.7. Graphs showing more realistically achievable economic benefits from using high-strength concrete in columns when the value of free floor space is also taken into consideration.

From the results presented above, the economic benefits of using HSC for main load bearing columns in high-rise buildings are now much more distinct and perceivable. The overall economic savings per column, when using HSC compared to the NSC C35/45, ranges from 25-56 % when one square meter of free floor space was valued at 2000 €/m^2 and 40-76 % when the floor value was set at 5000 €/m^2. Although these results indicate that very substantial profits can be made by using HSC instead of NSC, the absolute increased floor space due to smaller columns might not be as useful and as valuable as completely removing one row of columns or increasing the gap between columns. Thus, this estimation is admittedly a simplification regarding how much actual free floor area can be obtained and effectively utilized by using higher strength concrete. However, as the average floor prices in Helsinki for multi-storey residential buildings in 2018 were around 5000 €/m^2, the results from this comparison should not be considered overly optimistic. Additionally, the aspects of architecturally more appealing thinner columns and generally more durable structures connected with the use of HSC is not yet taken into consideration in any way. It can thus be expected that when all aspects of using HSC in large construction projects can be accurately evaluated and applied in practice, the overall benefits of using this material is even greater than this evaluation reveals.

Furthermore, as the reinforcement ratio of columns rises extensively it is worth mentioning that the use of other than the conventional steel reinforcement bars option should be considered. High-strength steel and larger steel sections, such as tubes filled with concrete or different steel profiles encased in concrete columns, are today widely used in high-rise buildings. These column designs, especially when combined with high-strength concrete, have been proven to be an effective way of constructing load bearing columns that are particularly useful in very high-rise buildings (Shanmugam and Lakshmi, 2001). Similarly as using HSC instead of NSC, these developed ways of utilizing steel sections, and, especially high-strength steel, for composite structures also supports decreasing the total weight of the building. But since the behavior of these kinds of columns differ considerably from conventionally reinforced concrete columns, they were not evaluated further in this study.

5 Experimental work

The purpose of the experimental work was to verify the appropriate behaviour of the concrete classes compared earlier. The testing was also intended to expose differences in behaviour between the different concrete classes, especially between the normal-strength concrete and high-strength concretes. The concretes to be produced were to represent the same concrete classes as compared in the previous chapter. In other words, one widely used normal-strength concrete class as reference, C35/45, and three high-strength concretes of different strength classes, C55/67, C70/85 and C90/105.

5.1 Composition of test specimens

In order for the tested concretes to be feasible concretes for the Finnish construction industry, the recipes for the concretes C35/45, C55/67 and C70/85 were provided by Rudus Oy, a Finnish building materials producer. As higher strength concretes are uncommon in Finland, the recipe for the concrete to represent the class C90/105 concrete was simply created by reducing the mixing water of the C70/85 mix design and increasing the superplasticizer dosage to reach the desired workability. However, the classification of this concrete as C90/105 concrete was only an initial speculation, based mainly on the w/cm ratio, on what strength range this concrete might prove to be in. The purpose of making this experimental higher strength concrete by simply reducing the water content was mainly to verify the impact the w/cm ratios have on the compressive strength of concrete. Additionally, it provided a valuable insight in how this change altered the overall behaviour of the fresh concrete and how effective modern superplasticizers truly are.

Since the aggregate used for the experimental work in this thesis differs from what Rudus Oy uses, the dosage of the superplasticizer had to be optimized in order to obtain similar workability as in the original concrete mix design made by Rudus Oy. While Rudus Oy at this time used Finnsementti's Parmix-Silika for their concretes containing silica fume, the concretes requiring silica fume in this thesis were made with Elkem Microsilica 920ED. Both mentioned silica fumes are made and tested according to the standard EN 13263-1. Otherwise the ingredients used were the same as in the original recipes. A summary of the materials used for the experimental work is shown in Table 5.1 and the chemical composition of the clinkers of the two cement types presented in Table 5.2.

Table 5.1. Summary of the materials used for the experiments (Finnsementti, 2019).

Material	**Name**	**Manufacturer**	**Specifications**
Cement	Mega-cement	Finnsementti Oy	• CEM I 52,5 N • Initial setting time, 170-230 min • Fineness (Blaine), 380-420m^2/kg • Loss of ignition, 2.1-2.4 %
	SR-cement	Finnsementti Oy	• CEM I 42.5 N - SR3 • Initial setting time, 160-200 min • Fineness (Blaine), 310-390m^2/kg • Loss of ignition, 1.9-3.6 %
Superplasticizer	Saitti-Parmix	Finnsementti Oy	• Polycarboxylate ether (PCE) based high range water reducing admixture • Normal dosage, 0.3-2.0 % of cement weight
Silica fume	Microsilica 920ED	Elkem AS	• Produced in accordance to the standard EN 13263 regarding silica fuma for concrete
Fly ash	Category A	Fly ash from Virkkala	• Fulfiling the requirements set for category A fly ash according to the standard EN 450-1

Table 5.2. Chemical composition of the cements used for the experiments (Finnsementti, 2019).

Chemical properties of cement clinker	Mega-cement [%]	SR-cement [%]
CaO	60-61	64-66
SiO_2	18-19	20-22
Al_2O_3	5.0-5.2	3.1-3.7
Fe_2O_3	3.1-3.2	3.9-4.2
MgO	4.3-4.6	2.7-3.5

As can be seen from Figure 5.1a, the Mega-cement has a slightly faster and higher strength development than the SR cement, likewise the hydration heat of the Mega-cement is also slightly higher. This justifies the choice of SR cement for the concretes with a higher cement content, Classes C70/85 and C90/105, in order to keep the hydration heat at a reasonable level. However, as indicated in Figure 5.1b, when purely high strength is required, the use of Valko-cement must be considered the best option.

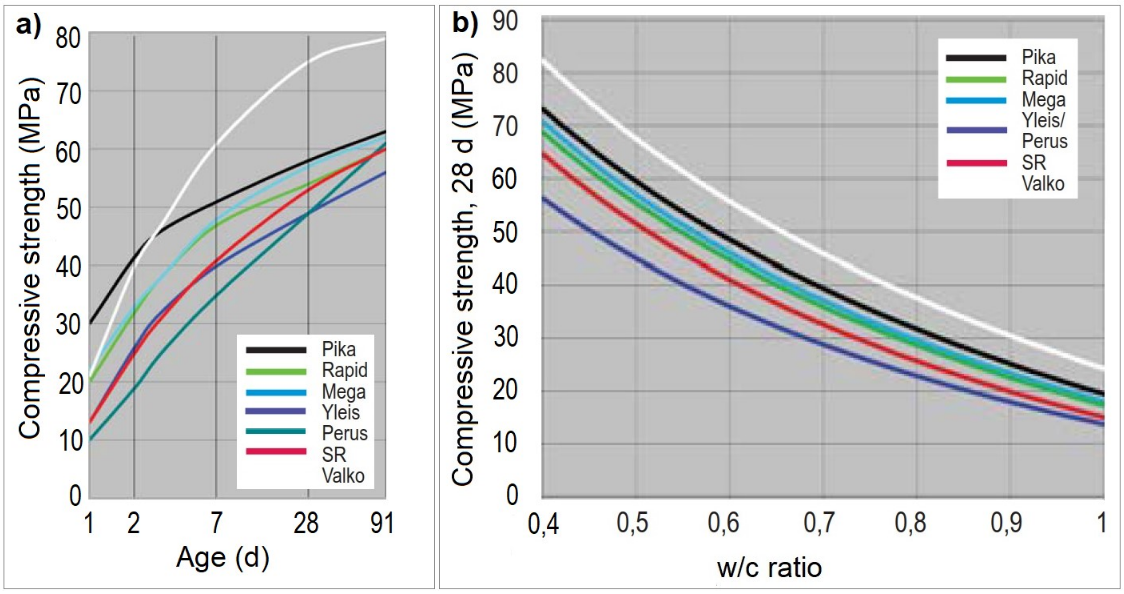


Figure 5.1. a) The strength development of different cement types available in Finland and b) how the cement type affects the 28-day compressive strength of the concrete at different w/c ratios (Finnsementti 2012).

The aggregate used for the experimental work in this thesis was composed of seven different aggregates available in the concrete laboratory at Aalto University, all natural and uncrushed. This in contrast to Rudus Oy, that use four different aggregates, one of which crushed. Therefore, a gradation test was performed on all seven of the laboratory aggregates in order to combine the aggregates and match the particle size distribution given in the original recipes by Rudus Oy. The aggregates used in the testing ranged from filler to 16 mm and were divided in the following seven classes; filler, 0.1-0.6 mm, 0.5-1.2 mm, 1-2 mm, 2-5 mm, 5-10 mm and 8-16 mm. The results of the gradation tests and aggregate combination can be seen in Appendix 1. Even though the size distribution can be matched well, concretes made by different aggregates may still behave dissimilarly due to differences in aggregate properties, such as absorption and surface area. However, the slight differences are mainly seen through an increased or decreased water need for the concrete. This consequently takes form in workability variations. Therefore, the possible variation in the behaviour of the concretes due to the use of different aggregates was minimized by the superplasticizer optimization process for the laboratory created concretes where everything except the superplasticizer dosage was kept unchanged.

The final concrete compositions, after the optimization of superplasticizer dosage, are shown in Table 5.3a. The combined aggregate of each concrete was made to match the original gradation curves of the aggregates used by Rudus Oy in the original recipes. The final composition of the combined aggregates consisting of the available aggregates fractions in the concrete laboratory is shown in Table 5.3b. All aggregate details and combined aggregate gradation curves plotted against the upper and lower limits, according to the Nykänen proportioning method for concrete with 16 mm maximum aggregate size, can be seen in Appendix 2.

Table 5.3. a) Composition of the concretes studied. b) Composition of the final combined aggregate for each concrete class.

a)

	C35/45 [kg/m³]	C55/67 [kg/m³]	C70/85 [kg/m³]	C90/105 [kg/m³]
Cement	299*	419*	518**	518**
Silica fume	-	-	27	27
Fly ash	84	118	-	-
Aggregate	1812	1670	1696	1696
Superplasticizer	*1.017*	*3.143*	*6.009*	*9.583*
Water	188	187	183	160
SP/C	*0.34 %*	*0.75 %*	*1.16 %*	*1.85 %*
w/cm	*0.57*	*0.40*	*0.32*	*0.28*

** Mega-cement*
*** SR-cement*

b)

	Aggregate fraction [mm]	C35/45 % of total aggregate	C55/67 % of total aggregate	C70/85 % of total aggregate	C90/105 % of total aggregate
Filler		11 %	8 %	8 %	8 %
Fine	0.1-0.6	7 %	10 %	9 %	9 %
	0.5-1.2	12 %	6 %	8 %	8 %
	1-2	12 %	15 %	13 %	13 %
Coarse	2-5	15 %	16 %	17 %	17 %
	5-10	15 %	16 %	16 %	16 %
	8-16	28 %	29 %	29 %	29 %

All the aggregates used were dry. However, no extra water was added to counteract the aggregates absorption. Thus, the effective mixing water would be slightly lower than what is expressed in the Table 5.3a above. This was not seen as a problem since any possible workability losses due to the aggregates water absorption would still be minor and the effects minimized by the superplasticizer dosage optimization process.

The w/cm ratio presented in Table 5.3a includes the performance factor of the supplementary cementitious materials. By multiplying the silica fume and fly ash content by a k-value of $k_{SF} = 2.0$ and $k_{FA} = 0.4$, respectively, and adding these to the cement content, a general value for the cementitious materials is obtained. Thus, the w/cm ratio was calculated according to the following Equation 20 below;

$$^{w}/_{cm} = \frac{W}{C+k_{SF}*SF+k_{FA}*FA} \tag{20}$$

where W is the mixing water, C is the cement content, SF is the silica fume content and FA is the fly ash content.

5.2 Experiment procedures

In the following sections the applied procedures connected to producing and testing of the studied concrete classes are described.

5.2.1 Mixing

The mixing of all the concrete batches was done in a Pemat Zyklos ZZ75HE rotating pan type of mixer with a capacity of 120 kg. The addition of each dry constituent material into the mixer followed the sequence shown in Figure 5.2. The mixing started with 30 seconds mixing of only dry materials. Thereafter about 90 % of the mixing water was added and mixed for one minute. After this the superplasticizer combined with the remaining water was added and the concrete was mixed for another three minutes.

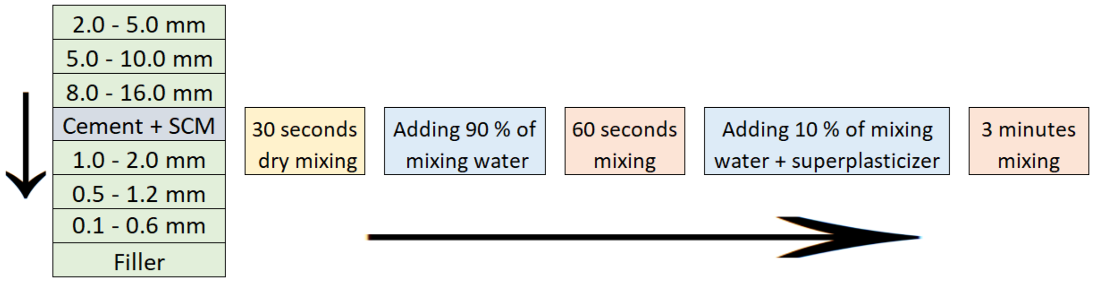


Figure 5.2. Material adding sequence and mixing durations.

The required mixing time for high w/cm ratio-concretes are likely to be lower than that of low w/cm-ratio-concretes. Therefore, the mixing time could probably be optimized for each concrete class, but for overall comparability reasons the mixing times presented above were considered appropriate and kept constant for all concrete classes. However, a sufficient mixing time, to assure that the concrete ingredients get properly mixed even in a drier mixture, is a crucial part in accomplishing the design strength of the concrete, especially for high-strength concretes.

5.2.2 Workability

The easiest, but perhaps not always the best, way of measuring the workability of fresh concrete is the slump test. The slump test according to EN 12350-2:2009, was used extensively in the practical testing part of this thesis. Not only for the final batches of each concrete, but also in the process of optimizing the superplasticizer dosage. In the original recipes provided by Rudus Oy, the designed slump class was S3, equalling 100-150 mm, for this work a target slump of 150 ± 25 mm was therefore decided as satisfactory. From Table 5.4, the optimization process and resulting optimal SP dosage for each concrete class can be obtained.

Table 5.4. The process of optimizing the superplasticizer dosage to reach the target slump. Optimization batches being 30 dm³ and the final batches 40dm³.

	C35/45:			C55/67:			C70/85 I:			C90/105:		
	SP [g]	SP/C [%]	Slump [mm]	SP [g]	SP/C [%]	Slump [mm]	SP [g]	SP/C [%]	Slump [mm]	SP [g]	SP/C [%]	Slump [mm]
Optimization batches	67.2	0.75	245	75.4	0.60	75	139.9	0.90	55	318.6	2.05	220
	35.9	0.40	175	100.6	0.80	185	194.3	1.25	155	303.0	1.95	190
	30.0	0.34	150	94.3	0.75	150				287.5	1.85	145
Final batches		**0.34**	**150**		**0.75**	**135**		**1.25**	**190***		**1.85**	**175**
							C70/85 II:	***1.16***	***175***			

** Too high slump, resulting in another batch, named C70/85 II, with SP/C = 1.16 % and a slump of 175 mm*

Although the raw materials and mixing process during the testing period was kept unchanged, it was noticed how easily affected the slump values can be, even by seemingly minor variables. These variations in slump became most perceptible during the mixing and casting of the final batches of the two highest strength concretes. At the final casting the concretes, C70/85 and C90/105, with the previously established optimal SP dosage showed an unexpected increase in slump. The C70/85 concrete had a slump of 190 mm and had to be re-mixed with a lower SP dosage to reach the target slump, whereas the C90/105 concrete got a slump of 175 mm and was assessed borderline satisfactory. These discrepancies, compared to the optimizing trial batches, are likely to be a result of the combination of an increase in temperature and humidity in the laboratory, different cement batches used and a slight change of batch size. The actual air-content is another factor that could affect the slump, but since no air-entraining was used and the mixing procedures were unchanged, this should not be the cause to the changes in slump in this case. However, small batches like these, 30-40 dm^3, are always more sensitive to even the slightest changes in the mixing procedures and environmental factors than larger concrete batches.

5.2.3 Air content

Since the concretes evaluated in this study were designed as non-air-entrained concretes without any air-entraining agents the measurement of the actual air content of the fresh concrete mixes was considered unnecessary. Even so, measurements would have been useful to confirm that the fresh concrete mixes did not have an unexpected and undesirable air content, over 20 dm^3/m^3. However, the presence of undesirable excess entrapped air seems unlikely since the measured densities or the test specimens concurred with the theoretically calculated densities based on the densities of the constituents of the concrete along with the assumption of an air content of 2 %.

5.2.4 Casting and curing

Of each concrete class, twelve samples were cast into 100 mm steel and plastic cube moulds for the compressive strength tests. The first batch of C70/85 concrete, which showed the unexpected and too high slump value, was nevertheless cast into the cubes required for the compressive strength testing, thus resulting in the label C70/85 I. The plastic moulds were only used for the second batch of the C70/85 concrete, subsequently called C70/85 II. All the 100 mm cube specimens were wet cured in a humid room, RH = 95 %, after the demoulding which took place roughly 24 hours after casting. During the first day in the moulds the specimens were covered by a plastic sheet to minimize the evaporation.

For the temperature development monitoring of each concrete class, an additional specimen was cast into a polystyrene form to represent the circumstances in the core of a large concrete column. The samples in the polystyrene moulds were kept in a normal indoor environment with an ambient temperature of approximately 22 °C.

5.2.5 Temperature development

Although problems due to the hydration heat of concrete is mainly associated with mass concrete, the use of high cement content concretes, such as high-strength concrete, may also cause concern in other types of structures. Especially when the dimensions of structural elements, such as columns, necessarily grow relatively large due to great loads, as demonstrated in Chapter 4.1. Therefore, a testing procedure to monitor temperature changes in the concretes studied in this thesis was implemented to investigate whether concrete structures made with concretes like these, may result in harmful temperature levels. The testing was mainly designed for recognizing the magnitude of maximum temperatures that might develop in the core of a massive reinforced high-strength concrete column. Another interesting aspect enabled by this test was to analyse the variations of the temperature development and scale between the peak values of the different concrete classes tested. As the two lower grade concretes incorporated fly ash as a SCM compared to the two higher grade concretes which contained silica fume instead, it was also expected that the difference this should have on the concretes heat developments could be recognized.

The isolating moulds used to cast the concrete samples consisted of a polystyrene box, including a lid, additionally insulated with 50 mm of polystyrene on all outside surfaces, as can be seen in Figure 5.3. The original polystyrene boxes having 22 mm thick walls and the inside measurements of 218 x 188 x 154 mm^3 resulted in concrete specimens of 6,3 dm^3 cast into them. This amount of concrete was considered being a well big enough sample for the purpose. While the insulating layer was probably not sufficient for reaching the absolute maximum temperatures of the concretes, it was thought adequate for realistically representing the temperature behaviour at the centre of a column structure made with the studied concrete.

Figure 5.3. Picture of the isolating mould used for monitoring he temperature development of the different concrete classes. On The right, the additional insulation into which the original polystyrene box, left, was placed.

Directly as the concrete had been mixed, it was placed in the mould and a sensor placed in the middle of the concrete sample. The sensors were connected to a Pico TC-08 thermocouple data logger which recorded the temperatures at an interwall of one minute for at least 6000 minutes for each concrete batch.

5.2.6 Strength development

Compressive strength tests at the ages 3d, 7d, 28d and 91d were scheduled in order to evaluate the variations in strength development between the different concrete strength classes. This was done mainly to confirm the differences between normal and high-strength concretes strength, but also to examine whether the use different cements and SCMs would result in any noticeable variations. Another interesting aspect would be how the experimental

C90/105 named concrete would perform and how much simply reducing the mixing water can improve the strength of concrete.

Three concrete samples at the different test ages, for each concrete, was assumed to be sufficient for the purposes of these tests. For the test specimens the 100 mm cube size was chosen due to its more convenient handling properties and because the concretes to be tested were of a relatively high strength, made with 16 mm maximum aggregate size. The different mould types used, shown in Figure 5.4, would prove to have an interesting effect on the compressive strengths of the specimens and will be discussed further in the following chapter.

Figure 5.4. The moulds used for casting the 100 mm cube specimens for the compressive testing.

5.3 Results and discussion

The results from the temperature monitoring and compressive strength tests are presented and discussed in the following sections along with analyses on why these results were obtained and what they may indicate.

5.3.1 Temperature development

The results from the temperature development monitoring tests are presented in Figure 5.5. The maximum temperature reached by the concretes C35/45, C55/67, C70/80 and C90/105 were 44.4, 55.1, 60.9 and 60.3 °C, respectively. As expected, the three high-strength concretes showed a significantly higher maximum temperature than the normal-strength concrete. Additionally, the resemblance between the temperature curves of the C70/85 and C90/105 concrete classes was also completely predictable due to the identical amount of cement in the two concretes. This would suggest that very high-strength concretes do not cause more trouble concerning excessive temperature rise during curing than moderate high-strength concrete. This heat generation is after all mainly governed by the cement content and type, not the w/cm ratio.

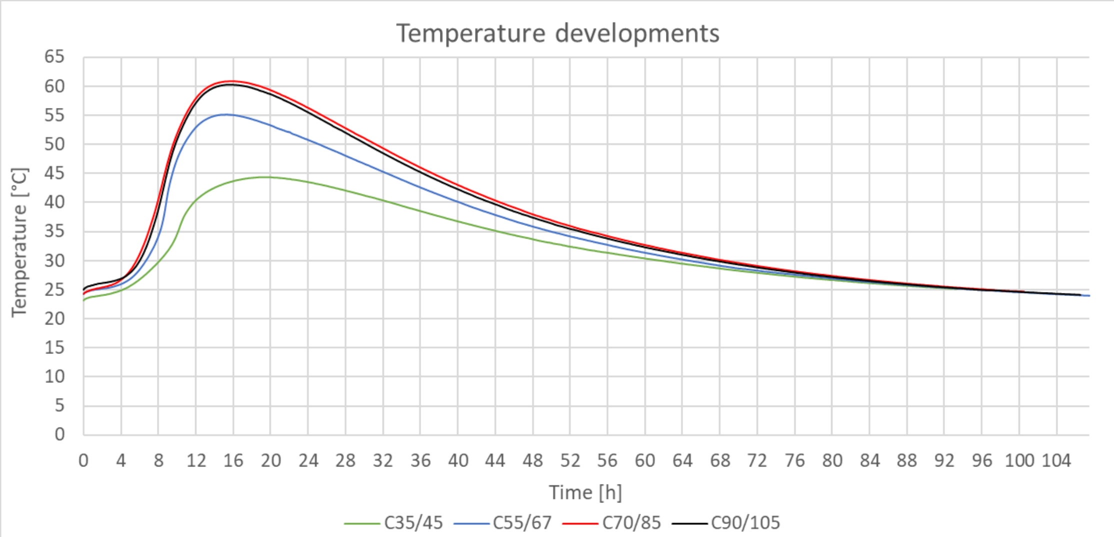


Figure 5.5. Results from the temperature development monitoring of each concrete class studied.

Considering that the concretes C35/45 and C55/67 contains fly ash, which is known to reduce the heat generation of concrete, at a replacement ratio of about 22 % of the cement, especially the temperature of the C55/67 concrete is noticeably close to the higher strength concretes which has a considerably higher cement content and also containing silica fume. However, this might be explained by different types of cements used, the higher strength concretes were indeed made using the slightly less reactive SR cement.

In comparison to the laboratory testing, the temperature development of four column cases, familiar from Chapter 4, were modelled by Rudus Oy in the BetoPlus software. The columns modelled were those from the 30 MN load-case applying the minimum reinforcement principle, with varying cross-section dimensions corresponding to the concrete class used. The cross-sections modelled with the resulting maximum temperatures are listed in Table 5.5 and the corresponding temperature curves are shown in Figure 5.6. Additionally, the accuracy of the modelling by BetoPlus can be considered being around ± 5 °C.

Table 5.5. The maximum temperatures obtained for a sample of column cases modelled in the BetoPlus software by Rudus Oy.

Concrete strength class	Column cross-section dimensions b [mm]	x	h [mm]	Maximum temperature [°C]
C35/45	1200	x	1150	53.0
C55/67	1000	x	950	63.7
C70/85	900	x	900	75.6
C90/105	850	x	850	74.5

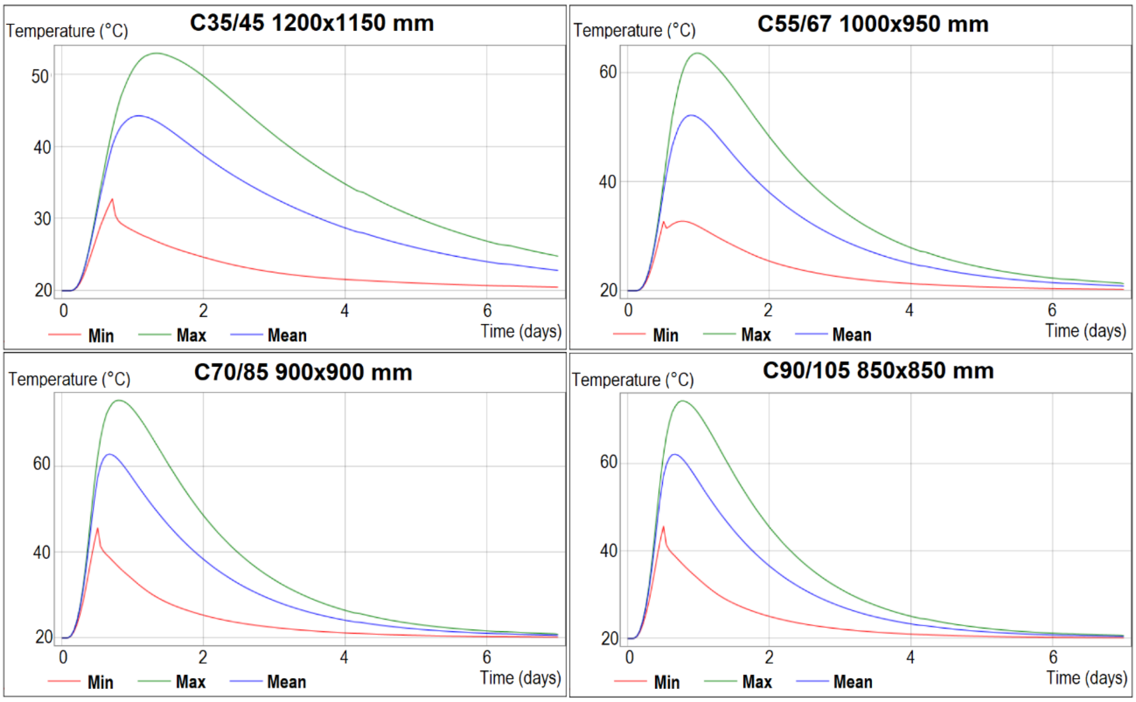


Figure 5.6. Resulting temperature curves from the BetoPlus modelling of the temperature development for a set of studied column cases.

As the results show, significantly higher temperatures were obtained from the modelled column cases than from the practical tests. This indicates that the experimental testing practises and results should not be relied upon as an accurate representation of the actual maximum temperatures at the core of columns with similar dimensions as mentioned before. Therefore, especially when massive columns made by high cement-content HSC are designed, the significant temperatures caused by the hydrating cement during the curing of the concrete columns should be taken into serious consideration and actions for minimizing the harmful effects this can have on the concrete should be evaluated. However, as the shapes of the curves of the modelled temperature development and the experimental test results are similar, the experimentally obtained temperatures presumably correspond better with the actual temperature development at the centre of less massive cast in-situ concrete columns.

5.3.2 Strength development

The results of the compressive strength tests are presented in the Table 5.6 and Figure 5.7. These values are the mean value of three 100 mm cube specimens at each test age, a full record of the test results can be seen in appendix A.

Table 5.6. The mean compressive strength at the different test ages for each concrete class studied.

Age [d]	C35/45 [MPa]	C55/67 [MPa]	C70/85 I [MPa]	C70/85 II [MPa]	C90/105 [MPa]
3	29.8	44.7	58.9	56.9	68.8
7	38.2	54.3	69.1	64.0	78.1
28	47.9	65.4	82.0	77.0	93.1
91	58.4	79.3	88.4	84.2	100.2

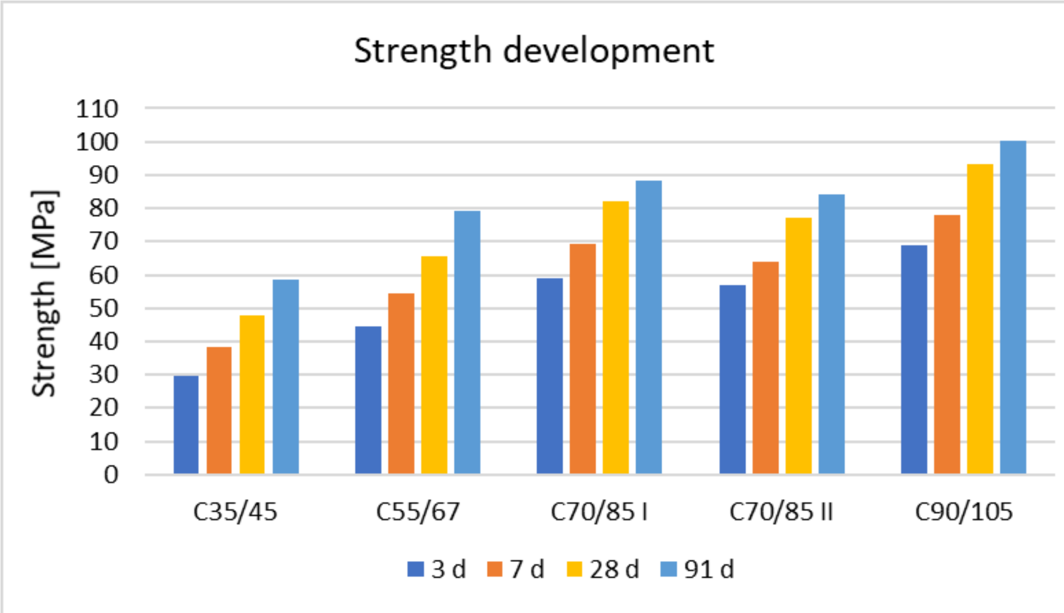


Figure 5.7. Graph showing the compressive strength development of the studied concrete classes at each specified test age.

To transform the compressive strengths of the tested 100 mm cube specimens to commonly recognised 150 mm cube compressive strengths a correction factor, k_{corr}, should be applied according to the Equation 21 below;

$$f_{ck.cube150} = \frac{f_{ck.cube100}}{k_{corr}} \tag{21}$$

According to the Finnish concrete norms (BY 65, 2016), the correction factor to be applied in this case has the value, k_{corr} = 1.03. The correction factor may vary depending on which standard is followed but it is commonly recognized that the strength of 150 mm cubes are a few percent lower than that of 100 mm cubes, according to Neville (2011) the 150 mm cube has 96 % of the strength of a 100 mm cube.

As can be concluded from the resulting compressive strength values, the requirements for the initial supposed strength classes of the concretes are not fulfilled at the age of 28 days by any of the specimens tested in this study. Especially when considering that a safety margin based on the standard deviation of the compressive test results should be added as well. However, the inferior results are likely caused by multiple factors that could be improved by utilizing optimized production methods and equipment.

Firstly, the mixing procedures for such small batches as was produced in this study were probably not optimal, especially for the drier concretes. It was noticed that at the end of the mixing, a layer of un-mixed dry materials remained densely packed at the bottom rim of the mixing drum. This could have been especially significant for the higher strength concretes containing silica fume since the materials stuck in the pan appeared to consist mostly of fine particles resembling silica fume to the colour. To ensure proper mixing and full incorporation of the relatively small amount of silica fume into the concrete mixture, the addition of silica fume should perhaps have been done as a slurry with a small amount of the mixing water instead of as plain silica fume powder. Alternatively, the total mixing time of the concrete should have been increased or a more optimal sequence of adding the materials investigated. However, this problem is most likely due to the small batches produced in this work, where even small amounts of unblended silica fume or cement can have a significant effect on the end results. Thus, this is an unlikely problem for concrete producers with modern equipment, optimized mixing procedures and producing large concrete batches.

Secondly, the consolidation of the concrete specimens might not have been ideal and could have been investigated more thoroughly in order to achieve higher compressive strengths in the compression tests. The cube samples for the compression tests were compacted using a vibrating table for only a couple of seconds. This was assumed adequate since the concretes were relatively workable, with slumps of 150 ± 25 mm. However, since no air content measurements were done at the casting occasion to verify that the concretes did not contain a higher percent of air than normal in non-air-entrained concretes, the vibration time could perhaps have been longer to ensure no additionally entrapped air was present in the concrete specimens. Nevertheless, the vibration time should not be exaggerated either, since this could lead to segregation and bleeding. The density of the test specimens measured at the time of the compression tests did not however indicate that the concretes contained any unexpectedly high percentages of air. As can be seen from Figure 5.8, the theoretically calculated densities of the concretes, assuming an air content of 2 %, were all lower than the measured mean densities for the concrete samples for the compressive strength tests.

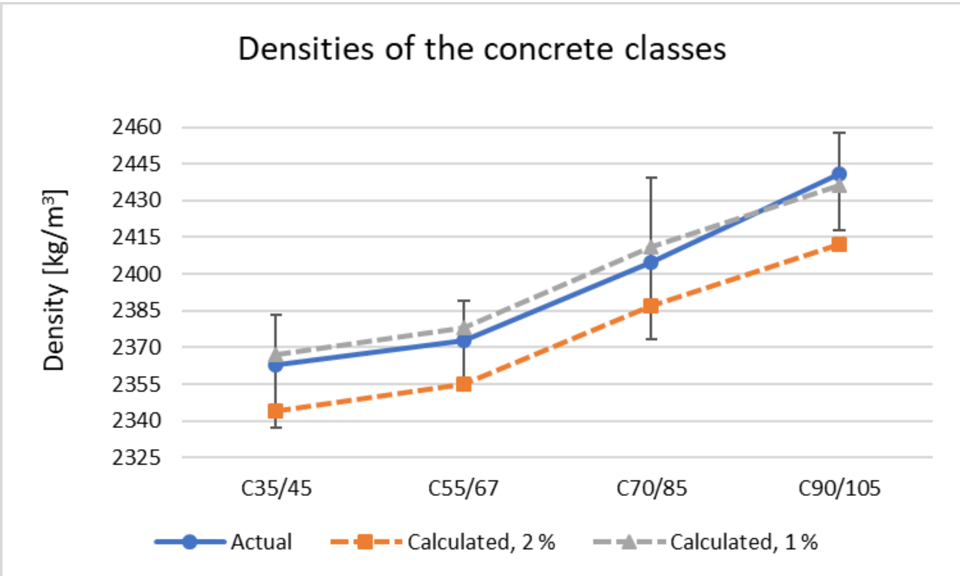


Figure 5.8. The mean densities with the extreme low and high values measured for the compression test samples against the calculated densities assuming an air content of 1 and 2 %.

As can be seen from the graph above, the measured densities for the concrete specimens matches fairly well the calculated densities when an air-content of only 1 % was assumed. This indicates that the actual air-contents of the tested concretes were closer to 1 % than 2 %. The fact that even the extreme values of the measured densities, as plotted in the diagram above as the error margin, barely fell below the calculated densities, assuming an air-content of 2 %, for only a few specimens also indicates that excess air-content was not a problem in the tests. Additionally, it can be mentioned that the standard deviation of the densities for all the concrete samples of each concrete class, C35/45, C55/67, C70/85 and C90/105, were

1.37 %, 1.02 %, 1.84 % and 1.01 %, respectively. All exact values for the measured densities at the compression tests can be seen in Appendix A.

Although the compressive strengths of the concretes did not, or barely reached the minimum strength values connected with each of the initially proposed concrete strength classes, even after 91 days, the testing showed that high-strength concrete classes are by no means unobtainable. Even the optimistically labelled C90/105 concrete class, which was a simply modified version of the C70/85 only with less mixing water, showed promise that even strength classes above 100 MPa are fairly easily achievable. The author is thus confident that by optimizing the production procedures according to the available equipment, the minimum strength values for at least the classes C35/45, C55/67 and C70/85 are reachable. For the concrete C90/105 to confidently reach the minimum compressive strength of that class, minor modifications to optimize the mix design is probably necessary. However, this shows that if even in laboratory conditions the fulfilment of the design strengths is no certainty, the correct handling of the concrete mixture, from mixing to curing, is of utmost importance for achieving hardened concrete satisfying the initial design properties.

6 Conclusions

The aim of this thesis was to evaluate the use of high-strength concrete and investigate how to most effectively reach the full potential of this material. The results from the literature review indicates that HSC is primarily used for high-rise buildings but also for particular structures where the requirements on the concrete and its performance are very demanding. Such structures can include bridges, off-shore structures and power stations, or more generally, structures that require a high durability. Thus, it can be assumed that the trend of building higher and higher has been the main reason for the increased use of HSC. However, studies also indicate that HSC can be used in more common building projects to enable more economical and environmentally sustainable building designs.

The comparison conducted in this thesis, on the effects of using different concrete strength classes for highly loaded reinforced concrete columns in a high-rise building, strengthens the previously stated assumptions regarding the benefits of using HSC. The comparison where a hypothetical high-rise building project with realistic values and variables was investigated to evaluate how the use of different concrete strength classes affects the dimensions and total costs of the main load-bearing columns of the building. Between the four concrete strength classes evaluated, C35/45, C55/67, C70/85 and C90/105, it was clearly seen that the three high-strength classes were in all load cases more cost-effective than the normal-strength concrete option. However, the initial economic benefits of using HSC only in the range of 3-11 % decreased column costs. The differences between the three high-strength concrete classes varied depending on load case, but no specific strength class could be chosen as the generally most optimal.

Although the economic benefits of using HSC did not appear very significant from the preliminary results of the column comparison, as presented in Chapter 4, some central advantages of using HSC were initially left unevaluated due to the difficulties of valuing them accurately. These aspects were mainly the increased free floor space and better durability connected to the use of HSC. The increased free floor area obtained by using C55/67, C70/85 and C90/105 concrete classes resulted on average in 33 %, 43 % and 49 % smaller column cross-section areas, respectively, compared to the NSC column option. These results indicate that the economic benefits of increased free floor space due to the use of higher strength concretes should be considered a significant variable for the final costs of RC columns. Thus, this aspect was also attempted to realistically implement into the comparison by fixing floor prices at both a low, 2000 €/m^2, and a high value, 5000 €/m^2. As the additional free floor area obtained by using HSC was multiplied with the mentioned floor values, a significant reduction in final column costs for the higher strength concrete column options could be observed in all cases. When the valuation of floor area was added to the evaluation, the resulting final costs of the HSC columns were 25-56 % and 40-76 % less expensive than the corresponding NSC column options, when the floor area was valued at 2000 €/m^2 and 5000 €/m^2, respectively.

Another purpose of this thesis was to investigate the uncertainties and problems connected to the use of HSC. These uncertainties are mainly related to the freeze-thaw resistance, fire endurance and workability issues of HSC. It is fair to assume that the frost durability of HSC is indeed better than normal concrete. However, it should not be assumed that HSC without additional air-entraining is per definition totally frost resistant. Although most studies indicate that even non-air-entrained HSC exhibits good frost resistance, especially as the w/cm ratio drops towards 0,3 and below, cases exist where HSC has shown poor freeze-thaw durability. Regarding the fire resistance of HSC on the other hand, the literature and studies are unanimous that it is indeed poorer than for regular concrete. Nevertheless, once the poor fire resistance of HSC is acknowledged there are numerous means to effectively overcome it, the most convenient in many cases is to incorporate polypropylene fibres to the concrete mixture. The reputation of HSC being less workable than ordinary concrete is perhaps a bit vaguer than the earlier mentioned problems. Undoubtedly the high cement contents and small amount of mixing water makes higher strength concretes "stickier" and more prone to significant workability losses if not properly designed and handled. Even so, with the constant progresses in superplasticizer and chemical additives industry this problem seems to be surmountable with adequate testing and quality control of new high-strength concrete mixtures.

Finally, this thesis aimed to examine how easily achievable the higher strength concretes are and whether excess hydration heat can be a problem for HSC columns. The same concrete strength classes as compared in the column comparison in Chapter 4 were produced in the laboratory of the University. Twelve 100 x 100 x 100 mm^3 specimens of each concrete class were cast to study the strength development by performing compression tests at the ages of 3, 7, 28 and 90 days. From the results of the compression tests it was acknowledged that none of the produced concretes met the requirements for the characteristic strength at 28 days, and only the classes C35/45 and C55/67 exceeded the required strengths at 90 days. From this it can be concluded that concrete with high strengths can be challenging to reach without thoroughly optimizing the applied mix designs and procedures. However, as the initial mix designs were intended for large scale concrete production, and even the NSC concrete failed to reach the minimum strengths for its strength class at 28 days, it can be assumed that the mixing and casting conditions were not optimal and should have been investigated further to reach the full potential of the mix designs.

To examine the hydration heat of the concrete classes, the temperature development for 6.3 dm^3 concrete specimens of each class were kept in an insulated environment and monitored for 100 hours after the casting. The temperature monitoring was designed to model the conditions at the centre of massive RC columns made with the studied concrete classes. The obtained maximum temperatures of the concrete classes C35/45, C55/67, C70/80 and C90/105 were 44.4, 55.1, 60.9 and 60.3 °C, respectively. These results, in combination with even higher temperatures obtained from BetoPlus temperature development models concerning the same column cases, suggest that especially for massive HSC columns the hydration heat produced during the curing process can be significant. Thus, precautions for the concrete mix design and curing conditions should be considered to avoid harmful consequences from elevated temperatures of the concrete caused by cement hydration.

To further enable the use of high-strength concrete, more thorough studies should be conducted concerning the economic benefits of using HSC in high-rise buildings, but also for more common and moderately loaded structures. Additionally, the probable benefits of using high strength steel in combination with HSC should also be investigated further. Furthermore, conclusive research regarding the freeze-thaw durability of HSC would be valuable. However, the basic assessment conducted in this study regarding the economic viability of using HSC in highly loaded structures strongly indicates that the use of HSC for such purposes is indeed favourable. Therefore, the author can recommend further utilization of high-strength concrete, particularly in load bearing elements of high-rise buildings.

References

ACI CT-13. (2013). *ACI Concrete Terminology, an ACI Standard*. American Concrete Institute. www.concrete.org.

Aïtcin, P.-C. (1998). *High-performance concrete*. London: E. & FN Spon.

Aïtcin, P.-C. (2003). The durability characteristics of high performance concrete: A review. *Cement and Concrete Composites*. Vol. 25(4-5), p. 409–420. doi: 10.1016/S0958-9465(02)00081-1.

Akca, A. H., Zihnioğlu, N. Ö. (2013). High performance concrete under elevated temperatures. *Construction and Building Materials*. Vol. 44, p. 317–328. doi: 10.1016/j.conbuildmat.2013.03.005.

Aldred, J. (2010). Burj Khalifa – a new high for high-performance concrete. *Proceedings of the Institution of Civil Engineers.* Vol. 163(2), p. 66-73. doi: 10.1680/cien.2010.163.2.66

Alves, M.F., Cremonini, R.A., Dal Molin, D.C.C. (2004). A comparison of mix proportioning methods for high-strength concrete. *Cement and Concrete Composites.* Vol. 26(6), p. 613-621. doi: 10.1016/S0958-9465(03)00036-2

Breitenbücher, R. (1998). Developments and applications of high-performance concrete. *Materials and Structures*. Vol. 31, p. 209–215.

Büyüköztürk, O., Lau, D. (2004). High performance concrete: Fundamentals and Application. *Department of Civil and Environmental Engineering, Massachusetts Institute of Technology, Cambridge*.

BY 65 (2016). *BY 65, Betoninormit 2016*. Helsinki: Suomen Betoniyhdistys ry.

Caldarone, M. A., Taylor, P. C., Detwiler, R. J., & Bhidé, S. B. (2005). *Guide specification for high performance concrete for bridges*. Portland Cement Association, EB233, www. cement. org.

Carrasquillo R.L., Slate F.O., Nilson A.H. (1981). Microcracking and Behaviour of High Strength Concrete Subjected to Short Term Loading. *ACI Journal Proceeedings.* Vol. 78(3), p. 179-186.

Carbonari, B. M. T., Gettu, R., Agullo, L., Carbonari, G. (2003). Analysis of production factors in high performance concrete. *Semina: Exact and Technological Sciences*. Vol. 24(1), p. 49-58.

CCAA (2009). Chloride Resistance of Concrete. *Cement Concrete & Aggregates Australia report, June.* p. 1–37. doi: 10.1016/j.cemconcomp.2009.08.001.

Chandra, S. and Björnström, J. (2002). Influence of superplasticizer type and dosage on the slump loss of Portland cement mortars — Part II. *Cement and Concrete Research*. Vol. 32(10), p. 1613–1619.

Collepardi, M., Collepardi, S., Ogoumah Olagot, J. J., & Troli, R. (2003). Laboratory-tests and field-experiences of high-performance SCCs. *Proceedings of the Third International Symposium on Self Compacting Concrete, Reykjavik*, p. 904-912.

CTBUH (2019). Council on Tall Buildings and Urban Habitats, The Skyscraper Database. http://www.skyscrapercenter.com [Accessed 23.01.2019]

Cusson, D., Hoogeveen, T. (2008). Internal curing of high-performance concrete with pre-soaked fine lightweight aggregate for prevention of autogenous shrinkage cracking. *Cement and Concrete Research*. Vol. 38(6), p. 757–765. doi: 10.1016/j.cemconres.2008.02.001.

EN 1992-1-1 (2004). Design of concrete structures - Part 1-1: General rules and rules for buildings. CEN - European Committee for Standardization.

EN 1992-1-2 (2004). Design of concrete structures - Part 1-2: General rules-Structural fire design. CEN - European Committee for Standardization.

Erdoğdu, Ş. (2005). Effect of retempering with superplasticizer admixtures on slump loss and compressive strength of concrete subjected to prolonged mixing. *Cement and Concrete Research*. Vol. 35(5), p. 907–912. doi: 10.1016/j.cemconres.2004.08.020.

Erdoğdu, Ş., Arslantürk, C., Kurbetci, Ş. (2011). Influence of fly ash and silica fume on the consistency retention and compressive strength of concrete subjected to prolonged agitating. *Construction and Building Materials*. Vol. 25(3), p. 1277-1281. doi: 10.1016/j.conbuildmat.2010.09.024.

Farny, J. A., & Kosmatka, S. H. (1997). *Diagnosis and control of alkali-aggregate reactions in concrete*. Skokie, IL: Portland Cement Association.

Ferraris, C., F. (1995). Alkali-silica reaction and high performance concrete. NIST Inter-agency/Internal Report (NISTIR) - 5742.

fib (2012). *fib Bulletin 65: Model Code 2010, Final draft – Volume 1*. Lausanne, Switzer-land: International Federation for Structural Concrete (fib). ISBN 978-2-88394-105-2

Fidjestol, P. and Dåstol, M. (2008). The history of silica fume in concrete–From novelty to key ingredient in high performance concrete. *Congresso Brasilero do Concreto*. Elkem Materials, Norway.

Finnsementti (2012) *Suomalainen Sementti-Opas*. http://www.finnsementti.fi [Accessed 12.11.2018]

Finnsementti (2019). Finnsementti's product range and specifications of products. https://finnsementti.fi/tuotteet/ [Accessed 20.01.2019]

Gjørv, O. E. (2008) *High strength concrete. Developments in the Formulation and Rein-forcement of Concrete.* Chapter 3, High-strength concrete. p. 79-97. Woodhead Publishing. ISBN 9781845692636.

Hale, W. M., Freyne, S. F., Russell, B. W. (2009). Examining the frost resistance of high performance concrete. *Construction and Building Materials*. Vol. *23*(2), p. 878-888. doi:10.1016/j.conbuildmat.2008.04.006

Helland, S. (1997). Application of High-Strength Concrete in Norway. *ACI Special Publication, 167.*

Hertz, K. D. (2003). Limits of spalling of fire-exposed concrete. *Fire safety journal*. Vol. 38(2), p. 103-116.

Hi-con (2019). Hi-con, high-performance concrete elements company. https://www.hi-con.com/references [Accessed 24.01.2019]

Johari, M. M., Brooks, J. J., Kabir, S., Rivard, P. (2011). Influence of supplementary cementitious materials on engineering properties of high strength concrete. *Construction and Building Materials*. Vol. 25(5), p. 2639-2648. doi:10.1016/j.conbuildmat.2010.12.013

Kawai, T. (2002). State-of-the-art report on high-strength concrete in Japan–Recent developments and applications. *JSCE/VIFCEA, Joint Seminar on Concrete Engineering*.

Khaliq, W. (2012). *Performance characterization of high performance concretes under fire conditions.* Doctoral dissertation, Michigan State University. Available at: https://doi.org/10.25335/M58199

Khaliq, W., Kodur, V. (2017). Effectiveness of polypropylene and steel fibers in enhancing fire resistance of high-strength concrete columns. *Journal of Structural Engineering*. Vol. 144(3). ISSN 0733-9445.

Khan, M. I., Lynsdale, C. J. (2002) Strength, permeability, and carbonation of high-performance concrete. *Cement and Concrete Research*. Vol. 32(1), p. 123–131. doi: 10.1016/S0008-8846(01)00641-X.

Kodur, V. K. R., Cheng, F. P., Wang, T. C., Sultan, M. A. (2003). Effect of Strength and Fiber Reinforcement on Fire Resistance of High-Strength Concrete Columns. *Journal of Structural Engineering.* Vol. 129(2), p. 253–259.

Kodur, V. K. R. (2005). Guidelines for fire resistance design of high-strength concrete columns. *Journal of Fire Protection Engineering*. Vol. 15(2), p. 93–106. doi: 10.1177/1042391505047740.

Kodur, V. K. R. (2008). Strategies for improving the performance of high-strength concrete columns under fire hazard. *Structural Control and Health Monitoring*. Vol. 19, p. 88–106. doi: 10.1002/stc.

Kodur, V. K. R., Phan, L. (2007). Critical factors governing the fire performance of high strength concrete systems. Vol. 42(6-7), p. 482–488. doi: 10.1016/j.firesaf.2006.10.006.

Konkov, V. (2013). Principle approaches to high performance concrete application in construction. *Procedia Engineering.* Vol. 57, p. 589-596. doi: 10.1016/j.proeng.2013.04.075

Kukko, H., Tattari, K. (1995). Korkealujuusbetonien säilyvyys. *VTT julkaisuja 808.* Valtion teknillinen tutkimuskeskus, VTT, Rakennustekniikka.

de Larrard, F. and Sedran, T. (2002). Mixture-proportioning of high-performance concrete. *Cement and Concrete Research*. Vol. 32(11), p. 1699–1704. doi: http://dx.doi.org/10.1016/S0008-8846(02)00861-X.

Marushima, N., Kuroha, K., Tomatsuri, K., Kubota, K., Koibuchi, K., Ishikawa, Y. (1993). Study on high-strength concrete with blast furnace slag cement incorporating very fine slag. In *Proceedings of the symposium on utilization of high strength concrete,* Lillehammer, Norway. Vol. 2, p. 830-37.

Marzouk, H., & Langdon, S. (2003). The effect of alkali-aggregate reactivity on the mechanical properties of high and normal strength concrete. *Cement and Concrete Composites*. Vol. 25(4-5), p. 549-556. doi: 10.1016/S0958-9465(02)00094-X.

Mcfarlane, A. (2007). High-strength concrete columns: A design guide. *The Structural Engineer*. Vol. 85(6), p. 31–38.

Meeks, K. W., Carino, N. J. (1999). *Curing of high-performance concrete: Report of the state-of-the-art*. NISTIR report 6295, US Department of Commerce, Technology Administration, National Institute of Standards and Technology.

Mendis, P. (2003). Design of high-strength concrete members: state-of-the-art. *Progress in Structural Engineering and Materials*. Vol. 5(1), p. 1–15. doi: 10.1002/pse.138.

Mills, A. (2010). Analysis of the cost performance of structural building frames in Australia. *Proceedings of the 7th ICEC World Congress & 14th PAQS Congress: Sustainable Cost Management: Borderless Innovatoin,* p. 1-15. Singapore Institute of Surveyors and Valuers.

Nagataki, S. (1997). High-Strength Concrete in Japan: History and Progress. *ACI Special Publication*, *167*.

Naik, T. R., Kumar, R., Ramme, B. W., Canpolat, F. (2012). Development of high-strength, economical self-consolidating concrete. *Construction and Building Materials*. Vol. *30*, p. 463-469. doi:10.1016/j.conbuildmat.2011.12.025

Neville, A., Aitcin, P.-C. (1998). High performance concrete—an overview. *Materials and structures*. Vol. 31(2), p. 111-117.

Neville, A.M. (2011). *Properties of Concrete,* Fifth Edition. Harlow, England: Pearson Education Limited. ISBN 978-0-273-75580-7.

Paulik, P. (2013). The effect of curing conditions (in situ vs. laboratory) on compressive strength development of high strength concrete. *Procedia Engineering.* Vol.65, p. 113–119. doi: 10.1016/j.proeng.2013.09.020.

PERI (2018). *Formwork costs*. I. Roihuvuo, from PERI Suomi Ltd Oy, e-mail conversation 7.9.2018.

Phan, L. T., Carino, N. J. (2003). Code provisions for high strength concrete strength-temperature relationship at elevated temperatures. *Materials and Structures/Materiaux et Constructions*. Vol. 36(256), p. 91–98. doi: 10.1617/13811.

Pons, O., de la Fuente, A. (2013). Integrated sustainability assessment method applied to structural concrete columns. *Construction and Building Materials*. Vol. *49*, p. 882-893.

Punkki, J., Golaszewski, J., & Gjørv, O. E. (1996). Workability loss of high-strength concrete. *Materials Journal*. Vol. *93*(5), p. 427-431.

Rashid, M. A., Mansur, M. A. (2009). Considerations in producing high strength concrete. *Journal of Civil Engineering*. Vol. 37(1), p. 53–63. Available at: http://www.jce-ieb.org/pdfdown/3701005.pdf.

Rudus (2018a). *Realistic Concrete material costs*. M. Tulimaa, from Rudus Oy, e-mail conversation 01.10.2018.

Rudus (2018b). *Valmisbetonihinnasto Etelä-Suomi 2018*. Price list of available concrete types and related services provided by Rudus Oy. Available at: https://www.rudus.fi/hinnasto-ja-esitteet/hinnastot/betonihinnasto. [Accessed 27.07.2018]

Setunge, S. (1993). *Engineering properties of High-Strength Concrete*. Ph.D thesis, Monash University.

Shanmugam, N. E. Lakshmi, B. (2001). State of the art report on steel-concrete composite columns. *Journal of Constructional Steel Research*. Vol. 57(10), p. 1041–1080. doi: 10.1016/S0143-974X(01)00021-9.

Shieh, S. S., Chang, C. C., Jong, J. H. (2003). Structural design of composite super-columns for the Taipei 101 Tower. *Proceedings of International Workshop on Steel and Concrete Composite Constructions* (pp. 25-33).

Singh, H. (2017) *Steel Fiber Reinforced Concrete : Behavior, Modelling and Design*. Springer. doi: 10.1617/s11527-010-9596-6.

Snellings, R. (2016). Assessing, Understanding and Unlocking Supplementary Cementitious Materials. *RILEM Technical Letters*. Vol. 1, p. 50. doi: 10.21809/rilemtechlett.2016.12.

Structurae (2019a). Structurae, International Database and Gallery of Structures. https://structurae.net/structures/trianon [Accessed 26.01.2019]

Structurae (2019b). Structurae, International Database and Gallery of Structures. https://structurae.net/structures/helgeland-bridge [Accessed 24.01.2019]

Trägårdh, J., Lagerblad, B. (1994). Alkali-silica reaction in high strength concrete. *Proceedings of the RILEM International Workshop, Durability of High Performance Concrete, in Vienna.*

Wu, D., Sofi, M., Mendis, P. (2010). High Strength Concrete for Sustainable Condtruction. *Proceedings of International Conference on Sustainable Built Environment, ICSBE-2010, Kandy*. p. 434–442.

Yang, K. H., Mun, J. S., Cho, M. S. (2015). Effect of curing temperature histories on the compressive strength development of high-strength concrete. *Advances in Materials Science and Engineering*.

Yousry, M. M., Shikh, E. (2008). Economy of Using High Performance Concrete Economy of Using High Performance Concrete. *33rd Conference on Our World in Concrete & Structures, Singapore*.

Zhutovsky, S., Kovler, K. (2012). Effect of internal curing on durability-related properties of high performance concrete. *Cement and concrete research*. Vol. 42(1), p. 20-26.

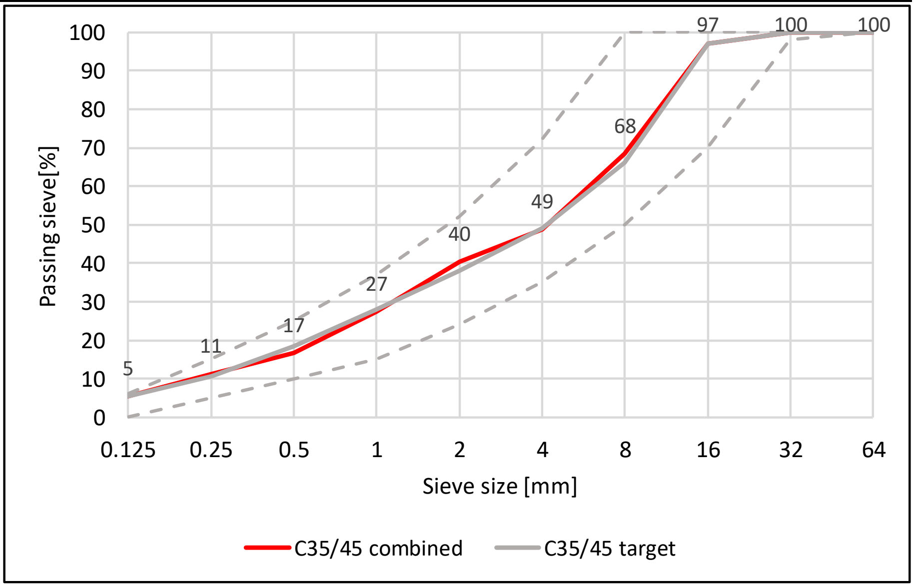
Appendices

Appendix 1. Gradation and combining of aggregates for tested concretes. 3 pages.
Appendix 2. Full record of compressive strength tests. 5 pages.

Appendix 1. Gradation and combining of aggregates for tested concretes.

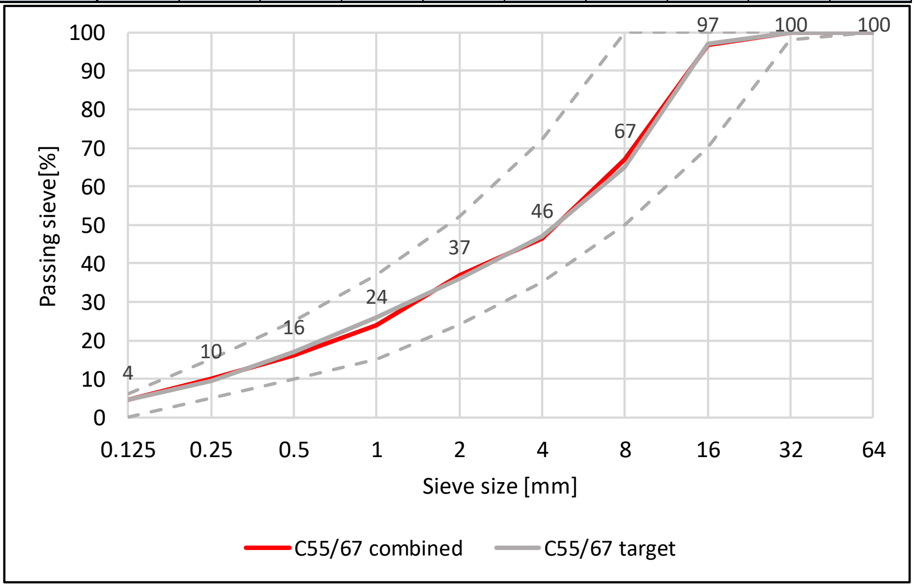
Gradation and combining of aggregate for concrete class C35/45

Fraction [mm]	Portion [%]	0.125	0.25	0.5	1	2	4	8	16	32	64
Filler	11 %	43	78	92	97	99	99	100	100	100	100
0.1-0.6	7 %	7	28	72	100	100	100	100	100	100	100
0.5-1.2	12 %	0	1	2	64	100	100	100	100	100	100
1-2	12 %	1	4	9	14	84	100	100	100	100	100
2-5	15 %	0	1	1	2	2	44	100	100	100	100
5-10	15 %	0	0	0	0	0	2	75	100	100	100
8-16	28 %	0	0	0	0	0	0	0	89	100	100
Combined [%]	100 %	5	11	17	27	40	49	68	97	100	100



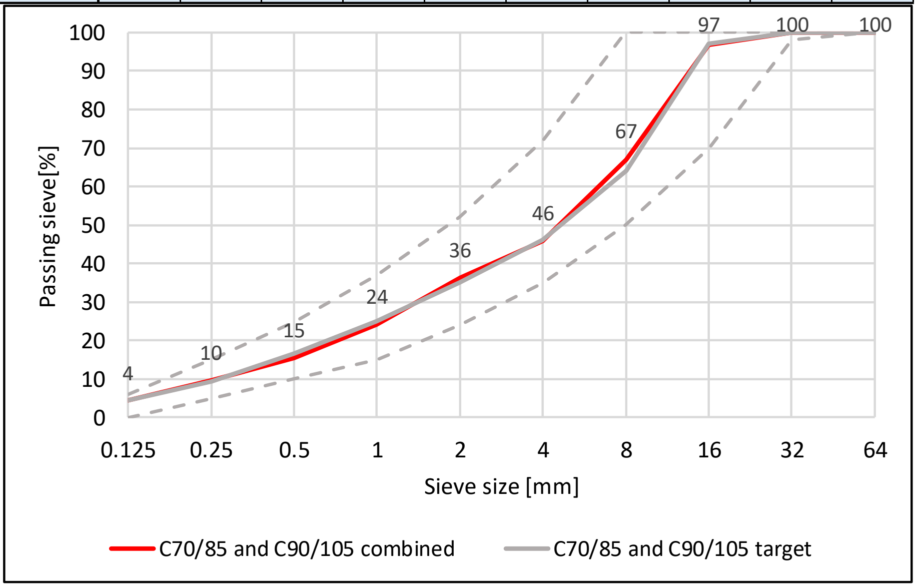
Gradation and combining of aggregate for concrete class C55/67

Fraction [mm]	Portion [%]	0.125	0.25	0.5	1	2	4	8	16	32	64
Filler	8 %	43	78	92	97	99	99	100	100	100	100
0.1-0.6	10 %	7	28	72	100	100	100	100	100	100	100
0.5-1.2	6 %	0	1	2	64	100	100	100	100	100	100
1-2	15 %	1	4	9	14	84	100	100	100	100	100
2-5	16 %	0	1	1	2	2	44	100	100	100	100
5-10	16 %	0	0	0	0	0	2	75	100	100	100
8-16	29 %	0	0	0	0	0	0	0	89	100	100
Combined [%]	100 %	4	10	16	24	37	46	67	97	100	100



Gradation and combining of aggregate for concrete classes C70/85 and C90/105

Fraction [mm]	Portion [%]	0.125	0.25	0.5	1	2	4	8	16	32	64
Filler	8 %	43	78	92	97	99	99	100	100	100	100
0.1-0.6	9 %	7	28	72	100	100	100	100	100	100	100
0.5-1.2	8 %	0	1	2	64	100	100	100	100	100	100
1-2	13 %	1	4	9	14	84	100	100	100	100	100
2-5	17 %	0	1	1	2	2	44	100	100	100	100
5-10	16 %	0	0	0	0	0	2	75	100	100	100
8-16	29 %	0	0	0	0	0	0	0	89	100	100
Combined [%]	100 %	4	10	15	24	36	46	67	97	100	100



Appendix 2. Full record of compressive strength tests.

C35/45, cast 17.07.2018

3 d compression test, 20.07.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	100.0	100.0	100.2	2.374	2.369	10000.0	301.0	30.10
2	99.7	99.7	100.2	2.368	2.378	9940.1	301.5	30.33
3	100.2	100.2	100.3	2.382	2.365	10040.0	292.1	29.09
				Mean:	**2.371**			**29.84**

7 d compression test, 24.07.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	100.4	100.4	100.0	2.376	2.357	10080.2	382.2	37.92
2	100.4	100.4	100.4	2.388	2.360	10080.2	397.2	39.40
3	100.4	100.4	100.0	2.381	2.362	10080.2	375.0	37.20
				Mean:	**2.360**			**38.17**

28 d compression test, 14.08.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	100.6	100.6	100.5	2.377	2.337	10120.4	470.9	46.53
2	100.6	100.6	100.1	2.414	2.383	10120.4	492.1	48.62
3	100.4	100.4	100.4	2.393	2.365	10080.2	487.9	48.40
				Mean:	**2.361**			**47.85**

91 d compression test, 16.10.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	99.7	99.7	100.6	2.383	2.383	9940.1	573.0	57.65
2	100.9	100.9	100.1	2.388	2.343	10180.8	598.0	58.74
3	100.2	100.2	100.5	2.375	2.354	10040.0	590.0	58.76
				Mean:	**2.360**			**58.38**

C55/67, cast 17.07.2018

3 d compression test, 20.07.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	101.1	101.1	100.4	2.436	2.374	10221.2	449.9	44.02
2	100.4	100.4	100.6	2.394	2.361	10080.2	451.1	44.75
3	100.4	100.4	100.6	2.415	2.382	10080.2	456.5	45.29
				Mean:	**2.372**			**44.68**

7 d compression test, 24.07.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	100.9	100.9	100.3	2.404	2.354	10180.8	549.0	53.92
2	100.0	100.0	100.8	2.394	2.375	10000.0	534.0	53.40
3	100.0	100.0	100.5	2.401	2.389	10000.0	555.0	55.50
				Mean:	**2.373**			**54.27**

28 d compression test, 14.08.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	100.9	100.9	100	2.403	2.360	10180.8	666.0	65.42
2	100.0	100.0	100.8	2.39	2.371	10000.0	642.0	64.20
3	100.7	100.7	100	2.419	2.385	10140.5	676.0	66.66
				Mean:	**2.372**			**65.43**

91 d compression test, 16.10.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	100.7	100.7	100.7	2.425	2.375	10140.5	792.0	78.10
2	100.4	100.4	100.8	2.419	2.381	10080.2	822.0	81.55
3	100.4	100.4	100.2	2.392	2.368	10080.2	790.0	78.37
				Mean:	**2.375**			**79.34**

C70/85 I, cast 23.07.2018

3 d compression test, 26.07.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	100.9	100.9	99.9	2.440	2.399	10180.8	609	59.8
2	101.4	101.4	100.6	2.455	2.373	10282.0	595	57.9
3	100.4	100.4	99.8	2.427	2.413	10080.2	596	59.1
				Mean:	**2.395**			**58.9**

7 d compression test, 30.07.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	100.2	100.2	100	2.426	2.416	10040.0	696	69.3
2	100.9	100.9	100.2	2.448	2.400	10180.8	694	68.2
3	100.0	100.0	100.6	2.454	2.439	10000.0	699	69.9
				Mean:	**2.418**			**69.1**

28 d compression test, 20.08.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	100.6	100.6	100.3	2.462	2.425	10120.4	830	82.0
2	100.9	100.9	100.3	2.435	2.385	10180.8	845	83.0
3	100.9	100.9	99.9	2.433	2.392	10180.8	826	81.1
				Mean:	**2.401**			**82.0**

91 d compression test, 22.10.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	100.0	100.0	100.5	2.448	2.436	10000.0	884	88.4
2	100.2	100.2	100.1	2.433	2.421	10040.0	892	88.8
3	101.2	101.2	100.2	2.436	2.374	10241.4	902	88.1
				Mean:	**2.410**			**88.4**

C70/85 II, cast 23.07.2018

3 d compression test, 26.07.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	99.4	99.4	99.7	2.383	2.419	9880.4	578	58.5
2	99.9	99.9	99.8	2.387	2.397	9980.0	555	55.6
3	100.2	100.2	99.6	2.389	2.389	10040.0	567	56.5
				Mean:	**2.402**			**56.9**

7 d compression test, 30.07.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	100.2	100.2	99.7	2.389	2.387	10040.0	639	63.6
2	99.9	99.9	99.9	2.404	2.411	9980.0	655	65.6
3	100.2	100.2	99.8	2.393	2.388	10040.0	630	62.7
				Mean:	**2.395**			**64.0**

28 d compression test, 20.08.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	100.6	100.6	99.8	2.413	2.389	10120.4	776	76.7
2	100.2	100.2	99.9	2.417	2.410	10040.0	774	77.1
3	100.0	100.0	99.7	2.406	2.413	10000.0	771	77.1
				Mean:	**2.404**			**77.0**

91 d compression test, 22.10.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	100.2	100.2	99.9	2.412	2.405	10040.0	842	83.9
2	99.9	99.9	99.8	2.406	2.416	9980.0	844	84.6
3	99.5	99.5	99.6	2.401	2.435	9900.3	832	84.0
				Mean:	**2.418**			**84.2**

C90/105, cast 23.07.2018

3 d compression test, 26.07.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	100.4	100.4	100.2	2.442	2.418	10080.2	683	67.8
2	99.9	99.9	100.5	2.465	2.458	9980.0	688	68.9
3	100.4	100.4	100.4	2.469	2.440	10080.2	703	69.7
				Mean:	**2.438**			**68.8**

7 d compression test, 30.07.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	99.7	99.7	100.5	2.436	2.438	9940.1	766	77.1
2	99.5	99.5	100.3	2.434	2.451	9900.3	790	79.8
3	100.4	100.4	100.6	2.468	2.434	10080.2	781	77.5
				Mean:	**2.441**			**78.1**

28 d compression test, 20.08.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	100.6	100.6	100.5	2.479	2.437	10120.4	950	93.9
2	100.7	100.7	100.5	2.48	2.433	10140.5	907	89.4
3	100.0	100.0	100.2	2.452	2.447	10000.0	959	95.9
				Mean:	**2.439**			**93.1**

91 d compression test, 22.10.2018:

Test specimen	l [mm]	b [mm]	h [mm]	Mass [kg]	Density [kg/dm^3]	Test face [mm^2]	Max load [kN]	Compressive strength [N/mm^2]
1	100.7	100.7	100	2.475	2.441	10140.5	1004	99.0
2	100.0	100.0	100.9	2.474	2.452	10000.0	1017	101.7
3	100.4	100.4	101.1	2.49	2.443	10080.2	1007	99.9
				Mean:	**2.445**			**100.2**