The use of high-quality recycled concrete aggregates in precast non-prestressed and prestressed concrete

Zeger SIERENS

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Preface

At the end of my PhD project, it is finally time to look back at the past 4.5 years. It has been a period full of surprises, meeting new people, a lot of ups and downs and most of all, continuously extending my boundaries. Many challenges crossed my way during the process, and it has not always been easy to overcome them. There were times when I did not see the end of the tunnel. Luckily I had a lot of support to help me evolve as a researcher and a person.

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Abstract

Nowadays, the use of recycled concrete aggregates (RCA) is mainly limited to non-structural concrete, although they have the possibility to be used in higher-valued applications. One of the main reasons for not using RCA in structural applications is the doubt to produce concrete with high and consistent performance. RCA have specific characteristics that influence the mechanical properties and durability aspects of concrete. RCA consist of the original aggregate, surrounded by attached mortar. This generally weaker and more porous attached mortar is considered one of the main reasons for the lower mechanical properties of concrete with RCA. Another reason for a conservative attitude towards RCA use in concrete is the heterogeneity of RCA. A proper classification system of different RCA types is necessary to reduce the heterogeneity and use them according to their distinct characteristics. RCA with a normal quality should be used in non-structural applications, while the high-quality RCA have the potential to be used in structural concrete with higher strength.

The high-quality RCA can reduce doubts about their use in structural concrete because of their high and consistent quality. The precast industry has already shown interest in using high-quality RCA in their products, but a lack of information on the specific influence of RCA withholds its implementation in precast concrete. Precast structural concrete demands additional requirements compared to non-structural concrete. The effect of RCA on early-age behaviour and variability of the mechanical properties of concrete, bond properties between reinforcement and RAC, and time-dependent properties need to be verified with industrially orientated research.

The main objective of this thesis is to assess the feasibility of the use of high-quality RCA in precast structural concrete and provide information on the influence of RCA on different aspects of precast concrete. The effect of RCA on different material properties, necessary for assessing precast structural concrete, is thoroughly investigated.

Firstly, the early-age behaviour of the mechanical properties is essential for the precast industry since this directly influences the production efficiency of a precast concrete plant. Changes in the early-age mechanical properties of concrete can influence the demoulding time and moment when the prestress releases. The results showed that incorporating RCA in precast concrete has a negative effect on the development of the compressive strength in the first three days after the concrete production. This effect can be minimised by limiting the amount of RCA to 20%$_{V/V}$. The high-quality RCA do not influence the strength development at 7 to 28 days. No apparent effect of RCA on the development of the tensile splitting strength and elastic modulus is observed. The curing condition has little effect on the concrete strength development when the coarse fraction of limestone is entirely replaced by the high-quality RCA. This contradicts the results from NAC, where a higher strength is observed for water-cured samples than air-cured samples.

Secondly, the variability of recycled aggregates has always been a significant issue as waste sources can be quite heterogeneous. On the other hand, the reliability of precast structural concrete products is particularly important since consistent quality is crucial in producing precast structural concrete. The results show that the variability of the compressive strength of precast concrete on a curing age of 1 day and 28 days increased when at least 50%$_{V/V}$ of the coarse limestone fraction is replaced by high-quality RCA. A replacement percentage up to 20%$_{V/V}$ did not result in increased variability. A decrease of 4% in compressive strength is observed when the high-quality RCA fully replace the coarse limestone fraction in the precast concrete mixture. A replacement percentage of up to 20%$_{V/V}$ does not statistically significantly change the compressive strength.

Thirdly, the bond behaviour between steel reinforcement and concrete with high-quality RCA is essential for any structural application since this influences the bearing capacity and the transfer length of reinforcement bars in concrete. It is found from the pull-out tests that the high-quality RCA do not have a considerable effect on the maximum bond stress of the specimens. However, the normalised bond strength, defined as the ratio of the maximum bond stress to the square root of the concrete compressive strength, slightly increases with the

increase of the RCA replacement percentage. A new empirical model is proposed to estimate the maximum bond stress. The model is formulated as a function of the concrete compressive strength and RCA replacement percentage. The proposed model predictions agree with the test data in the present study and literature.

Before using high-quality RCA in structural concrete, knowledge obtained from laboratory tests needs to be transferred to practice. Industrially produced samples have validated the lab tests on the early-age mechanical properties and variability of the compressive strength of precast concrete. Similar conclusions as for the lab tests are found in the analysis of the industrial tests. A general tendency of a slower strength development for precast concrete with RCA in the first three days after concrete production is confirmed. The difference in strength development becomes negligible at seven days. The difference in strength development for precast structural concrete with and without high-quality RCA is too small for any adaptions in the current conservative model predictions in Eurocode 2 [1] and ACI 209 [2]. The industrial tests can also confirm the conclusion made from lab tests that the precast structural concrete can replace up to $20\%_{V/V}$ of the coarse limestone fraction without significantly increasing the compressive strength variability.

The feasibility of using high-quality RCA in prestressed concrete is assessed by producing full-scale pretensioned prestressed concrete beams with different percentages of the high-quality RCA. The upward deflection (camber) of beams caused by releasing the prestress is measured and analysed. The measurements indicate that the presence of RCA results in an increasing initial camber of the prestressed beams. The lower elastic modulus of concrete with RCA causes the increased initial camber for precast concrete with the high-quality RCA. The relative evaluation of the long-term camber of the prestressed beams shows that a complete replacement of the coarse limestone fraction by the high-quality RCA results in an increased long-term camber by 20%. A replacement up to $20\%_{V/V}$ does not seem to affect the long-term camber development of the prestressed concrete beams.

Samenvatting

Tegenwoordig worden gerecycleerde betongranulaten voornamelijk gebruikt in niet-structureel beton, hoewel ze potentieel hebben om natuurlijke granulaten in betontoepassingen met hogere eisen te vervangen. Een van de belangrijkste redenen om geen gerecycleerde betongranulaten te gebruiken in structurele betontoepassingen is een vooringenomen twijfel om beton te kunnen produceren met consistente en voorspelbare eigenschappen. Gerecycleerde betongranulaten hebben specifieke eigenschappen die de mechanische en duurzaamheids-eigenschappen van beton beïnvloeden. Ze bestaan uit natuurlijke granulaten omgeven door aangehechte mortel. De zwakke en poreuze mortel wordt aanzien als een van de belangrijkste redenen voor de lagere mechanische eigenschappen van recyclagebeton. Een andere belangrijke reden voor een gereserveerde houding tegenover het gebruik van gerecycleerde betongranulaten is de hoge heterogeniteit. Een classificering van verschillende types gerecycleerde betongranulaten is nodig om de heterogeniteit te verminderen. Verschillende types gerecycleerde betongranulaten kunnen volgens hun kwaliteiten gebruikt worden in passende toepassingen. Gerecycleerde betongranulaten met mindere eigenschappen kunnen gebruikt worden in niet-structurele toepassingen, terwijl hoogwaardige gerecycleerde betongranulaten het potentieel hebben om gebruikt te worden in structureel beton met een hogere sterkte.

De goede en constante eigenschappen van hoogwaardige gerecycleerde betongranulaten kunnen eventuele twijfels wegnemen om dit materiaal in structurele betontoepassingen te gebruiken. De prefab industrie in Vlaanderen toonde reeds interesse om dit materiaal te gebruiken in hun producten, maar een tekort aan kennis naar de invloed van gerecycleerde betongranulaten op specifieke belangrijke eigenschappen voor de prefab industrie houdt het gebruik ervan tegen. Geprefabriceerd structureel beton heeft strengere eisen in vergelijking met niet-structureel beton. De invloed van hoogwaardige gerecycleerde betongranulaten op de sterkteontwikkeling van prefabbeton, variabiliteit van de eigenschappen van prefabbeton, binding tussen wapening en

prefabbeton en tijdsafhankelijke eigenschappen moet verder worden onderzocht met industrie-gerelateerd onderzoek.

Het belangrijkste doel van deze thesis is om de haalbaarheid van het gebruik van hoogwaardige gerecycleerde betongranulaten in geprefabriceerd structureel beton te beoordelen en om de invloed van dit type granulaten op verschillende eigenschappen gerelateerd aan prefabbeton te documenteren. Het effect van hoogwaardige gerecycleerde betongranulaten op eigenschappen die belangrijk zijn in de productie van geprefabriceerd structureel beton wordt diepgaand onderzocht.

Eerst en vooral wordt de sterkteontwikkeling van prefabbeton op jonge leeftijd behandeld. Deze kennis is essentieel voor prefabbeton aangezien het rechtstreeks de productie-efficiëntie in een prefabfabriek beïnvloedt. Een verandering in de sterkteontwikkeling van beton kan het tijdstip van ontkisten en het aanbrengen van de voorspankracht op het beton beïnvloeden. Uit de resultaten blijkt dat het gebruik van hoogwaardige gerecycleerde betongranulaten een negatief effect heeft op de sterkteontwikkeling in de eerste drie dagen na de productie van het beton. Dit effect is te verwaarlozen indien er slechts $20\%_{V/V}$ van de grove kalksteenfractie wordt vervangen door de recyclagegranulaten. De sterkteontwikkeling na 7 en 28 dagen wordt, zelfs bij een volledige vervanging van de grove fractie, niet beïnvloedt door de hoogwaardige gerecycleerde betongranulaten. Verder werd er geen invloed van de recyclagegranulaten op de ontwikkeling van de splijttreksterkte en elasticiteitsmodulus opgemerkt. De bewaaromstandigheden, d.i. in water en in lucht, hebben weinig effect op de sterkteontwikkeling van beton met hoogwaardige gerecycleerde betongranulaten, in tegenstelling met beton met natuurlijke granulaten, waar een hogere sterkte wordt behaald bij een bewaring van proefstukken onder water.

Vervolgens wordt de variabiliteit op de druksterkte van beton met hoogwaardige gerecycleerde betongranulaten bekeken. Een vermeende hogere variabiliteit in eigenschappen van recyclagegranulaten kan de betrouwbaarheid van het prefabbeton negatief beïnvloeden. Een constante kwaliteit is vanzelfsprekend van cruciaal belang in de productie van prefabbeton. Uit de resultaten blijkt

verder dat de variabiliteit op de druksterkte van prefabbeton stijgt indien er minstens 50%$_{V/V}$ van de grove fractie wordt vervangen, zowel op een ouderdom van 1 dag als 28 dagen. Vervangingspercentages tot 20%$_{V/V}$ hebben geen verhoogde variabiliteit in de druksterkte tot gevolg. Er wordt een druksterktedaling van 4% opgemerkt bij een volledige vervanging van de grove kalksteenfractie door hoogwaardige gerecycleerde betongranulaten. Een vervanging tot 20%$_{V/V}$ blijkt statistisch gezien geen invloed te hebben op de gemiddelde druksterkte.

Als derde wordt de hechting tussen de wapening en het prefabbeton met hoogwaardige gerecycleerde betongranulaten onderzocht. Deze hechting is essentieel voor beton in structurele toepassingen, aangezien het draagvermogen en verankeringslengte van de wapening in beton hier rechtstreeks door worden beïnvloed. Na het uitvoeren van verschillende 'pull-out' proeven blijkt dat de hoogwaardige gerecycleerde betongranulaten geen bepalend effect hebben om de maximale hechtsterkte van proefstukken met de wapening. Bij de bepaling van de genormaliseerde hechtsterkte, d.i. de verhouding van de maximale hechtsterkte en de vierkantswortel van de druksterkte, wordt een stijging van deze parameter bij een stijgend vervangingspercentage opgemerkt. Een nieuw empirisch model ter bepaling van de maximale hechtsterkte werd ontwikkeld in functie van het vervangingspercentage en de druksterkte van het beton. De voorspellingen van het voorgestelde model zijn in overeenkomst met de testresultaten in dit project en gelijkaardige projecten in gevonden literatuur.

Voordat hoogwaardige gerecycleerde betongranulaten effectief gebruikt worden, moet de kennis opgedaan in de uitgevoerde labotesten gevalideerd worden aan de hand van industriële testen. Er werden proefstukken geproduceerd in de prefab fabriek om de voorgaande labotesten op de sterkteontwikkeling en variabiliteit van de eigenschappen van beton met recyclagegranulaten te valideren. De resultaten van de industriële testen bevestigen de besluiten van de labotesten. De algemene tendens van een tragere sterkteontwikkeling in de eerste drie dagen na de productie van beton met recyclagegranulaten wordt bevestigd. Het verschil in sterkteontwikkeling na 7 dagen is te verwaarlozen. De verschillen in sterkteontwikkeling tussen beton met en zonder hoogwaardige gerecycleerde betongranulaten zijn klein genoeg zodat de opgemeten

sterkteontwikkeling nog steeds aan de conservatieve modellen uit de Eurocode 2 en ACI 209 voldoen. De industriële testen bevestigen tevens dat een vervangingspercentage tot 20%$_{V/V}$ van de grove fractie mogelijk is zonder een significante stijging in de variabiliteit van de druksterkte.

De haalbaarheid van het gebruik van hoogwaardige gerecycleerde betongranulaten in voorgespannen beton werd beoordeeld door het produceren van voorgespannen balken met verschillende percentages aan hoogwaardige gerecycleerde betongranulaten. De opbuiging van de balken onder invloed van de voorspanning werd opgemeten en geanalyseerd. De vervanging van natuurlijke granulaten door hoogwaardige gerecycleerde betongranulaten resulteerde in een stijging van de initiële opbuiging, voornamelijk veroorzaakt door een lagere elasticiteitsmodulus. Een relatieve vergelijking van de opbuiging van de balken op lange termijn toont aan dat bij een volledige vervanging van de grove fractie aan kalksteen door recyclagegranulaten de opbuiging stijgt met 20%. Een vervanging van 20%$_{V/V}$ van de grove fractie kalksteen heeft geen effect op de opbuiging van de voorgespannen balken.

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Definitions

AIR-CURING

Concrete samples in this project subjected to air-curing are cured in the open air with 20°C and relative humidity of 60-70%

CAMBER

The eccentricity of the prestressing force in prestressed beams causes an upward deflection of the beams. The upward deflection consists of an upward component and a downward component. The upward component is caused by the prestressing force and depends on the prestressing force, the eccentricity of the prestressing strands, and the concrete properties. The beams' self-weight will cause a downward component, depending on the dimensions and its material properties. The resultant deflection is calculated by subtracting the deflection due to the beams' self-weight from the deflection due to the prestressing force. In this project, the resultant deflection is called 'camber', which is the combination of the upward and downward components.

HIGH-QUALITY RCA

High-quality RCA are recycled concrete aggregates produced from pure and high-quality concrete rubble such as concrete of C50/60 and ready-mixed concrete. The recycling process is optimised, and two types of crushers are used to reduce the amount of attached mortar to the RCA. Generally, the WA_{24} of high-quality RCA can be estimated at around 4-5%.

INITIAL CAMBER

The initial camber is defined as the camber measured directly after the prestressing force is released on the concrete member.

LONG-TERM CAMBER

The long-term camber is defined as the total camber subtracted by the initial camber of the prestressed beam.

MEMBER-CURING

Concrete samples in this project subjected to member-curing are cured in the presence of a full-scale precast concrete member to recreate the conditions of the concrete member.

WATER-CURING

Concrete samples in this project subjected to water-curing are immersed in water basins of 20°C.

Notation

List of symbols

General

α_R	Contribution of the material resistance in the total variability at the design point
β	Reliability index
c	Distance from the reinforcement surface to the concrete surface
c_c	Distance from the reinforcement centre to the concrete surface
E_i	Expected frequency for interval i
d_b	reinforcement bar diameter
$fLTR$	From left to right
h_0	Notional height of a concrete beam
H_0	Null hypothesis
l	Embedded length
n	Number of samples
O_i	Observed frequency for interval i
σ	Population standard deviation
s	Sample standard deviation
s_{max}	Slip at the maximum bond stress
χ^2	Chi-square value
$\chi^2_{1-a,k-1-c}$	Critical chi-square value
γ_c	Partial safety factor for concrete
$\%_{M/M}$	Mass percentage

$\%_{V/V}$	Volume percentage

Composition and properties of recycled aggregates

ϱ_{RD}	Oven-dried particle density
ϱ_{a}	Apparent particle density
ϱ_{ssd}	Saturated and surface-dried particle density
Rc_{90}	Content of concrete products in recycled aggregates
Rcu_{90}	Content of concrete and natural stone in recycled aggregates
Rb_{10-}	Content of masonry rubble in recycled aggregates
Ra_{1-}	Content of bituminous materials in recycled aggregates
FL_{2-}	Content of floating materials in recycled aggregates (volume)
XRg_{1-}	Content of glass in recycled aggregates

Exposure and environmental classes of concrete

X0	Exposure class for no risk of corrosion or attack
XC1 to XC4	Exposure classes for risk of corrosion induced by carbonation
XD1 to XD3	Exposure classes for risk of corrosion induced by chlorides other than sea water
XS1 to XS3	Exposure classes for risk of corrosion induced by chlorides from sea water
XF1 to XF4	Exposure classes for risk of freeze/thaw attack
XA1 to XA3	Exposure classes for risk of chemical attack
E0	Environmental class for no aggressive environment
EI	Environmental class for internal environment
EE1 to EE4	Environmental classes for internal or external environment in contact with water

ES1 to ES4	Environmental classes for sea environment
EA1 to EA3	Environmental classes for aggressive chemical environment

Time-dependent properties of concrete

$\beta_{bc}(f_{cm})$	Basic creep component considering the compressive strength
$\beta_{bc}(t,t_0)$	Basic creep component considering concrete age
$\beta_{dc}(f_{cm})$	Drying creep component considering the compressive strength
$\beta(RH)$	Drying creep component considering the RH
$\beta_{dc}(t_0)$	Drying creep component considering concrete age at loading
$\beta_{dc}(t,t_0)$	Drying creep component considering age of concrete
$\beta_s(RH)$	Drying shrinkage component considering RH
$\beta_{ds}(t.t_s)$	Drying shrinkage component considering concrete age
$\beta_{bs}(t)$	Basic shrinkage component considering concrete age
$\varepsilon_{cs}(t,t_s)$	Total shrinkage strain
$\varepsilon_{cbs}(t,t_s)$	Basic shrinkage component
$\varepsilon_{cds}(t,t_s)$	Drying shrinkage component
$\varepsilon_{cbs0}(f_{cm})$	Basic shrinkage component considering compressive strength
$\varepsilon_{cds0}(f_{cm})$	Drying shrinkage component considering compressive strength
$\varphi(t,t_0)$	Creep coefficient
$\varphi_{bc}(t,t_0)$	Basic creep coefficient
$\varphi_{dc}(t,t_0)$	Drying creep coefficient
t_0	Age of concrete at loading
$t_{0,adj}$	Age of concrete at loading considering cement type
$t_{0,T}$	Age of concrete at loading considering temperature

t_s	Concrete age at beginning of drying in days

Mechanical properties of concrete

E_c	Static elastic modulus
E_{cm}	Mean static elastic modulus at 28 days
$E_c(t)$	Static elastic modulus at t days
$E_{cm}(t)$	Mean static elastic modulus at t days
f_c	Compressive strength of concrete, tested on cylinders
$\widetilde{f_c}$	Median compressive strength of concrete
$f_{c,max}$	Maximal compressive strength of concrete
$f_{c,min}$	Minimal compressive strength of concrete
f_{cm}	Mean compressive strength of concrete at 28 days, tested on cylinders
$f_{cm,cub}$	Mean compressive strength of concrete at 28 days, tested on cubes
$f_c(t)$	Compressive strength of concrete at t days, tested on cylinders
$f_{cm}(t)$	Mean compressive strength of concrete at t days, tested on cylinders
$f_{cm,cub}(t)$	Mean compressive strength of concrete at t days, tested on cubes
f_{ck}	Characteristic compressive strength
f_{cd}	Design compressive strength
$f_{ct,sp}$	Tensile splitting strength of concrete, tested on cubes
f_{ctm}	Tensile strength of concrete

$f_{ctm,sp}$	Mean tensile splitting strength of concrete at 28 days, tested on cubes
$f_{ct,sp}(t)$	Tensile splitting strength of concrete at t days, tested on cubes
$f_{ctm,sp}(t)$	Mean tensile splitting strength of concrete at t days, tested on cubes
$\tau_{b\ max}$	Maximum bond stress
$\tau_{b\ max,n}$	Maximum normalised bond stress

List of acronyms

CDW	Construction and demolition waste
CoV	Coefficient of variation
HES	High-early strength
HSC	High-strength concrete
ITZ	Interfacial transition zone
LA-value	Los Angeles Value
LVDT	Linear variable differential transformer
MRA	Mixed recycled aggregates
NA	Natural aggregates
NAC	Natural aggregate concrete
RA	Recycled aggregates
RAC	Recycled aggregate concrete
RCA	Recycled concrete aggregates
RMA	Recycled masonry aggregates
SCC	Self-compacting concrete

RASCC	Recycled aggregate self-compacting concrete
SLS	Serviceability limit state
StDev	Standard Deviation
ULS	Ultimate limit state
WA_{24}	Water absorption after 24h of immersion
W/C_{eff}	Effective water to cement ratio

Chapter I

Introduction

1.1 Recycling of concrete

Strong demographic growth over the last decades has resulted in a worldwide expansion of urban areas and industrialisation [3]. This phenomenon has undoubtedly resulted in growing pressure on natural resources. The construction industry is one of the largest consumers of natural resources, accounting for around half of the total use of resources [4]. The urban situation in Flanders, Belgium, causes a high demand for natural mineral resources in new buildings but also leads to depletion of them. Consequently, most natural aggregates need to be imported from surrounding regions resulting in high transportation costs, high greenhouse gas emissions and high impact on the environment. For this reason, it is of major importance to use all materials as efficiently as possible, while sourcing for local alternatives.

Construction and demolition waste (CDW) is, on the other hand, one of the largest generated waste streams in Flanders, accounting for 25-30$\%_{V/V}$ of all locally generated waste streams [5]. This waste stream covers a broad range of materials like concrete, bricks, tiles, wood, glass, metals, and others. Nearly a third of the total amount of CDW is concrete [6]. Recycled concrete debris could serve as a replacement for natural aggregates in concrete. This would result in a highly effective and promising approach towards a circular economy in the construction sector.

Concrete debris can originate from different sources, each with its specific properties. Demolition of buildings can produce a large amount of pure concrete

debris, assuming that a selective demolition protocol is followed. Tearing down a building and collecting all the waste streams to send them to landfills is not done anymore. Selective demolition becomes a more common practice in the deconstruction process of buildings, as the EU Construction & Demolition Waste Management Protocol [7] prescribes. In this way, every waste stream is separated, leading to an effective recycling process of pure concrete waste from foundations, floors and walls, and reinforced building elements. The demolition of infrastructure components is also a significant source of concrete debris. In this case, large quantities of concrete debris can be produced in a short time. This type of concrete debris is, in most cases, only slightly contaminated. The recycling of top layers of roads needs extra attention since contamination with de-icing salts must be avoided. Other concrete debris sources are production and construction errors and leftovers from the precast concrete production. These limited amounts of concrete debris used to be landfilled, but they are now one of the most-wanted sources of concrete debris due to their purity and high quality.



Figure 1-1 – Sources of concrete waste: construction and demolition waste, infrastructure waste and production errors in precast concrete (fLTR)

Research on recycled concrete aggregates (RCA) in construction applications was initiated by researcher Glushge in Russia in 1946 [8]. This was the start of many research pieces during the following years to examine potential applications of this material. The first initiatives in Flanders started between 1970 and 1980, when several articles were published describing different techniques to demolish and crush concrete structures [9]. The first breakthrough for the recycling industry in Flanders was in 1981 when the Waste Decree officially considered CDW a useful waste material to be recycled [10]. A few

years later, recycled concrete was applied to construct the Berendrecht-Lock in Antwerp [11]. More initiatives on recycling waste materials started when Luc Boehme funded RecyCon in 1998. RecyCon is a research group associated with KU Leuven, which specialises in the development of sustainable material management in the construction industry. In 1999, on behalf of the Belgian Building Research Institute, RecyHouse was constructed with only recycled materials [12]. Due to the academic research work of several research institutes and in close collaboration with the construction industry, governments, clients, and contractors started to accept recycled materials in road applications. Small percentages of natural aggregates were allowed to be replaced by RCA, but the use of recycled materials was mostly limited to foundation and sub-foundation layers of road work. In 2012, the project ValRecon20 was intended to showcase the possibility of using RCA in higher replacement percentages in strength classes up to C25/30 [13]. These results were adopted to allow larger replacement percentages in Belgian standard NBN B15-001 [14].

Although many interesting and stimulating initiatives concerning the use of RCA in construction applications have been organised in the last few decades, there are still several obstacles to overcome before using more RCA in the construction industry. One of the main reasons for the conservative attitude towards RCA use in concrete is producing it with predictable performance. The heterogeneity of RCA causes a lack of confidence.

1.2 Properties and use of RCA

RCA have specific characteristics that influence the mechanical properties and durability aspects of concrete. RCA generally consist of the original aggregate, surrounded by attached mortar, as shown in Figure 1-2. The mortar is less dense than the virgin aggregates, influencing the density and water absorption (WA_{24}) of the RCA. More attached mortar results in a lower density and a higher WA_{24}. The quality of the concrete debris influences the properties of the RCA. The quality of the attached mortar increases and, consequently, the WA_{24} decreases as the strength of the parent concrete increases. The amount of attached mortar is also highly dependent on the crushing process. Adding a secondary crushing

step in the recycling procedure will decrease the amount of attached mortar on the RCA compared to using a one-step crushing process. The larger WA_{24} of RCA influences its use in concrete. To obtain the correct W/C-ratio (W/C_{eff}), the WA_{24} of the RCA must be compensated by adding extra absorption water. When the absorption water is fully absorbed, the resulting free water remains equal to a similar mix with natural aggregates.

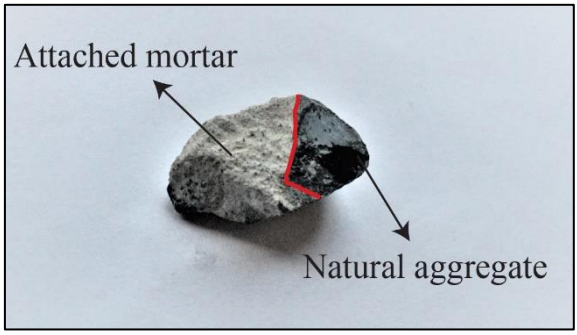


Figure 1-2 – Recycled concrete aggregate

The interface between cement paste and aggregates in concrete is one of the weaker zones in the complete concrete matrix for concrete with natural aggregates. This zone is called the interfacial transition zone (ITZ). The incorporation of RCA in concrete adds an additional ITZ to the concrete. The interface between the old attached mortar and the old natural aggregate within the RCA is the first ITZ, while the zone between the RCA and the new mortar is the second ITZ. The additional ITZ, possibly with extra micro-cracks resulting from the crushing process, can decrease the mechanical properties of concrete with RCA. In addition, the porous mortar is also more sensitive to pulverisation when loaded. RCA with lower amounts of attached mortar from concrete debris with higher strength have a better mechanical performance.

The properties of concrete change when using RCA because of the different properties of RCA compared to natural aggregates. The density of recycled aggregate concrete (RAC) is generally slightly lower than natural aggregate concrete (NAC) since the density of RCA is lower than that of the natural aggregates (NA) [15]. The influence of RCA on the mechanical properties of

concrete is described by many researchers [16]. The compressive strength decreases when increased replacement percentages of RCA are used in concrete with a similar composition. The degree of the strength loss depends on the quality, size, and replacement percentage of RCA. Reductions in compressive strength between 20 – 30% are expected for replacement percentages up to 100% [17–19]. Replacement percentages up to $30\%_{V/V}$ of the coarse fraction seem to have no significant effect on the compressive strength [20]. The tensile splitting strength is less affected by the RCA than the compressive strength. A stronger bond between the new cement paste and the old attached mortar of the RCA improves the ITZ's microstructure in the concrete. This improved bond results in an improvement of the tensile splitting strength. The elastic modulus generally decreases with the incorporation of RCA in concrete. The lower concrete stiffness depends on the aggregates' elastic modulus, which is smaller for RCA due to the porous attached mortar.

Presently, nearly all types of RCA are used in non-structural applications. It is known that the effect of RCA on the compressive strength of concrete increases with a higher strength class, as shown in Figure 1-3. Due to its lower mechanical properties, the RCA becomes the weakest part in the concrete matrix with a higher quality of the mortar fraction in a higher strength class of concrete. The amount of strength loss is dependent on the replacement percentage and the quality of the RCA. A considerable discrepancy in the strength loss of RAC compared to NAC, as noticed in different publications, is due to the variation in the quality of used RCA [16]. This variation is one reason why there are some doubts about the full incorporation of RCA in structural concrete. A proper classification system of different RCA types is necessary to use them according to their distinct characteristics. RCA with a lower quality should be used in non-structural applications, while the high-quality RCA have the potential to be used in structural concrete with higher strength.

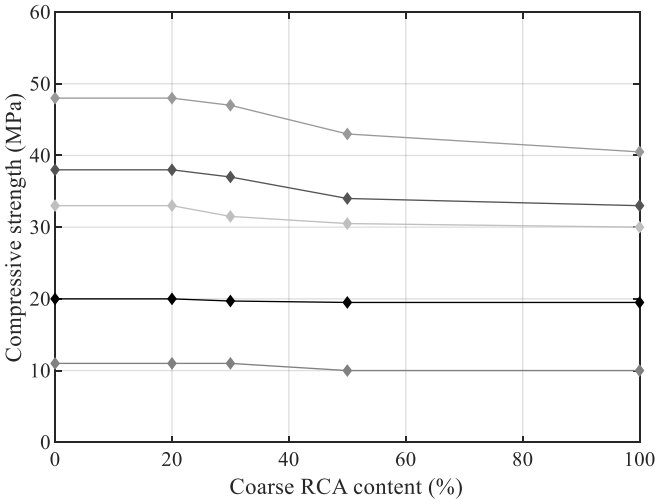


Figure 1-3 – Effect of replacement percentage of RCA on 28-day compressive strength (adapted from *[20]*)

1.3 RCA in structural concrete

The standards NBN B15-001 [14] and NBN EN 12620 [21] permit the use of recycled aggregates in different applications. The recycled aggregates are divided into two main categories: A+ and B+. RA A+ are RCA with a maximal water absorption of 10%, while RA B+ are mixed recycled aggregates with a maximal water absorption of 15%. According to their quality, the use of these types of recycled aggregates is allowed in different exposure classes, as described in NBN B15-001 [14] and section 2.1.2. RCA of type A+ with a maximal water absorption of 10% still covers a wide variation on RCA quality. They are not constant enough to be used in structural concrete with a high strength class.

When recycling concrete debris into aggregates for structural purposes, it is essential to create a constant high-quality RCA. Within the scope of this PhD research, high-quality RCA are defined with a maximal WA_{24} of 4-5%. This is only possible when concrete debris is completely separated from other waste during a selective demolition process. In this way, a pure concrete debris stream can lead to an effective recycling process. With an efficient sorting method,

different concrete debris qualities can be separated based on the type of original concrete and the purity of the waste stream. The high-quality RCA can be derived from the high-quality concrete waste, e.g., precast concrete waste and pure ready-mixed concrete. A second important factor in producing high-quality RCA is the recycling process. An optimised recycling process using two types of crushers can decrease the amount of attached mortar and result in high-quality RCA. It is reported that about 25% of the total amount of CDW produced in Flanders can be processed into high-quality RCA [22]. This type of material can be used in structural concrete and further promote the sustainability of the precast industry.

Until today, high-quality RCA are mainly used in non-structural concrete, while the structural behaviour of RAC has been investigated in several studies. The flexural properties are mostly tested on full-scale beams, where the crack pattern, ultimate bearing capacity and load-deflection behaviour were also studied [23–30]. Li [23] and Xiao et al. [24] gave an overview of different flexural tests performed in China. They conclude a similar crack pattern for beams with RCA and beams with natural aggregates. There is nearly no influence of RCA's incorporation on the beams' ultimate bearing capacity. However, the deflections, especially the immediate deflection, seem to be significantly higher for RAC beams. Similar results have been concluded by [25,26,29–31]. Pacheco et al. [27,28] reported no significant influence of RCA on the load-deformation curves. The authors concluded that the execution conditions have a larger influence on the structural behaviour than RCA's incorporation. Ignjatovic et al. [32] reported no significant differences in the service load deflections. The shear strength is an important parameter when evaluating concrete's structural behaviour. The concrete properties are more relevant when concrete beams are subjected to shear loads [24]. Gonzalez-Fonteboa et al. [31] reported decreased shear strength of concrete with RCA. However, the decrease is slight, and the prediction in codes are still conservative towards these results. Fathifazl et al. [33] reported similar results if the RAC is proportioned according to the described EMV method. Li and Xiao et al. [24] drew similar conclusions from different research work, although in the perspective of Li [23], the formulas in different codes tend to be unconservative for the shear strength of concrete with RCA.

Despite these promising results, one of the main reasons for not using RCA in precast structural applications at this moment is the uncertainty to produce concrete with RCA with consistent and predictable mechanical performance. Therefore, fundamental research is essential to resolve these uncertainties so that high-quality RCA can also be implemented in structural concrete.

1.4 RCA in precast structural concrete

The precast industry has already shown great interest in using high-quality RCA in their products, NBN EN 13369 [34] even allows small amounts of RCA in precast structural concrete elements when the effects of RCA in the precast mixture become clear by testing different aspects. This is only the case when the RCA are derived from a known origin or own precast products. However, precast producers in Flanders often do not have the ability to crush their concrete waste. In practice, the concrete waste is sent to an industrial crusher, where all pure concrete debris from different sources is collected and crushed into high-quality RCA ($WA_{24} \approx 5\%$). In this case, NBN EN 13369 only allows up to $10\%_{V/V}$ replacement if the required mechanical strength is tested on full-scale tests.

The production of precast concrete has different advantages. Concrete products can be produced independently of weather conditions. The controlled environment at the precast plant decreases any interruptions of the concrete production process and enhances the final element's quality. The concrete properties of the produced elements can easily be tested and controlled before placing the elements on the construction site. This enhances the concrete quality control. Given the conservative attitude towards producing concrete with RCA with predictable performance, a controlled environment with quality control such as at a precast plant is optimal to introduce high-quality RCA in structural concrete.

Recycling precast concrete has been a topic of research in some previous research projects. In 1998, a research project [35] led by BRE in association with several

precast contractors in the UK aimed to recycle rejected precast elements within newly produced precast elements. Several trials were run, replacing 20%$_{M/M}$ of the coarse aggregates and 10%$_{M/M}$ of the fine aggregates with RCA. Initial results indicate nearly no influences by implementing RCA in the precast structural element, as long as the replacement percentage is limited to 20%$_{M/M}$. In Belgium, a company manufacturing precast structural elements has also tested the influence of different RCA replacement percentages on the properties of fresh and hardened precast concrete [36]. Waste concrete generated by the production of its precast elements was used to produce the RCA. The results show no influence of the used RCA on the compressive strength for a replacement percentage up to 10%$_{M/M}$. A replacement up to 25%$_{M/M}$ only resulted in a slight decrease in compressive strength. Corinaldesi [37] reported preliminary results on the mechanical performance of concrete made by replacing crushed natural aggregate with RCA, discarded by prefabrication plant. The author reported that the quality of the original concrete influences the maximal replacement percentage. When the strength of the original concrete is 55 MPa, the replacement is possible for a percentage of 30%$_{M/M}$ of the total aggregate skeleton. Brandes and Kurama [38] published the results of a preliminary study on the effect of RCA on the properties of precast/prestressed concrete. The author tested the bond strength between steel strands and concrete, the development of compressive strength and elastic modulus in time. RCA from both rejected precast elements and ready-mixed concrete are used. The results showed that the RCA made from precast concrete yield better results than RCA made from ready-mixed concrete. The use of RCA did not significantly affect the strength development of concrete, even for a complete replacement of the coarse fraction of the aggregate skeleton. In the continuation of RCA research in precast concrete, Brandes and Kurama published the results of different tests on pre-tensioned beams with different RCA levels [39]. The authors concluded that the RCA did not affect the beams' service-load behaviour too much, even with a total replacement. They also stated that the current prediction models for immediate and sustained load deflections were not affected by RCA.

The above conclusions from previous research show the general technical possibilities to use RCA in precast structural concrete. Nevertheless, a lack of

information on specific influences of high-quality RCA concerning the use in precast concrete withholds its implementation in precast concrete. Precast structural concrete demands additional requirements compared to non-structural concrete. Precast reinforced concrete and precast prestressed concrete each have specific requirements that should be considered concerning the use of RCA. The effect of RCA on early-age behaviour of mechanical properties of concrete, the variability of the properties of precast concrete, bond properties between reinforcement and RAC, and time-dependent properties need to be verified with industrially orientated research.

1.5 Research objectives

The standard NBN EN 13369 [34] allows small percentages of RCA in precast applications if the feasibility is shown through systematic research. Within the scope of this PhD, the focus lies on evaluating the feasibility of using RCA in precast structural concrete and removing technical barriers to stimulate RCA use. Validation of reliable use of RCA in precast structural concrete can only be done by investigating the effects of RCA on the different properties of precast concrete.

The main objective of this thesis is to assess the feasibility of the use of high-quality RCA in precast structural concrete and to assess the influence of RCA on different aspects of precast concrete. The effect of RCA on different material properties, necessary for the assessment of precast structural concrete, is systematically investigated.

- First, **the early-age behaviour of the mechanical properties** is vital for the precast industry since this directly influences the production efficiency of a precast concrete plant. Changes in the early-age mechanical properties of concrete can influence the demoulding time and moment when the prestress releases. According to ACI 318, the prestress load can be transferred when the maximal concrete stresses do not exceed 60% of the maximum allowable compressive stress at the moment of prestress transfer [40]. Hence changes in the strength development of concrete can significantly affect the

production schedule of prestressed members. Current models used to predict the early-age behaviour of concrete are based on mechanical properties developed from testing conventional concrete with natural aggregates. This causes concern that the design of precast structural concrete incorporating high-quality RCA may not be accurate when equations developed for conventional concrete are applied. Different models will be developed based on experimental data to determine how the early-age behaviour of high-strength concrete with precast RCA can be predicted.

- Secondly, the variability of recycled aggregates has always been a significant issue as waste sources can be quite heterogeneous. The reliability of precast structural products is particularly important since consistent quality is crucial in producing precast structural concrete. Therefore, any **effect of RCA on the variability in compressive strength** of structural concrete needs to be investigated thoroughly and, if necessary, considered in the precast structural concrete design.
- Thirdly, **the bond behaviour between steel reinforcement and concrete with high-quality RCA** is essential for any structural application since this influences the bearing capacity and the transfer length of reinforcement bars in concrete. It is important to know how incorporating high-quality RCA in concrete influences the maximum bond strength and the bond-slip relation.
- Before using high-quality RCA in structural concrete, knowledge obtained from laboratory tests needs to be transferred to practice. The feasibility of using high-quality RCA in prestressed concrete will be assessed by **producing full-scale prestressed concrete beams** with different rates of high-quality RCA. The upward deflection (camber) of beams caused by the prestress load will be measured since this is an important issue for the precast industry. An overestimation or underestimation of the camber can cause problems in the service state of structures. To the best of the author's knowledge, no information on the influence of high-quality RCA on the camber of prestressed concrete beams is available in the literature. The knowledge from lab tests combined with the full-scale tests will give a

better understanding of the behaviour of RCA in precast structural concrete and will pave the way for the use of high-quality RCA in these applications.

The novelty of this research is found in the combination of industrially produced high-quality RCA incorporated in industrial precast concrete mixtures. The industrial precast mixtures are high-strength concrete with a target strength class of C50/60 since this is frequently required in precast plants to achieve an adequate early-age compressive strength of concrete. A knowledge gap is noticed between the extensive research using normal lab-made RCA in normal-strength concrete and actual practical situations. The practical use of high-quality RCA in precast structural concrete requires adapted and applied research with industrial conditions. The next steps in the actual use of high-quality RCA in precast products are only possible by removing any restrictions concerning the use of RCA through thorough research. This can be done by developing technical specifications on the different aspects of precast structural concrete influenced by the incorporation of RCA.

1.6 Research methodology

The methodology in this PhD thesis can be summarised into four stages.

- An analysis of current regulations and research on incorporating RCA in precast concrete has been done during the first stage. This information was researched in the published scientific literature.
- The second stage consists of the experimental programme, where various concrete tests were carried out on a laboratory scale. The main properties of the materials used in the tested industrial mixtures were thoroughly analysed. A laboratory study was set up to assess any changes in the concrete properties when the high-quality RCA replaced 0, 10, 20, 50 and 100%$_{V/V}$ of the coarse fraction 6.3-14 mm of limestone in the precast mixtures.
    - The early-age mechanical properties of concrete with the high-quality RCA were tested in the first test series. Tests with different curing conditions were designed to evaluate the

development of compressive strength, tensile splitting strength, and static elastic modulus on testing ages of 1, 2, 3, 7, 14 and 28 days.

  - The variability of the compressive strength of RAC was tested in the second series of tests. The strength variability of NAC was evaluated and compared with the variability in the strength of RAC. This evaluation was done statistically, and the influence on design considerations was determined.
  - The bond behaviour between RAC and steel reinforcement was tested in the third series of tests. Extensive pull-out tests were performed on the mixtures with different replacement percentages. The maximal bond strength and the bond-slip behaviour was thoroughly analysed. Based on the regression analysis of the test results, an adapted model was proposed to estimate the maximum bond stress between the steel rebars and concrete with RCA.

- The third stage includes validating the lab tests results by practical industrial tests. Different tests were done on an industrial level and compared with the laboratory tests. Industrial mixtures with different replacement percentages of high-quality RCA were produced in a concrete plant. These mixtures were used to produce prestressed concrete beams and to validate the test results of the early-age mechanical properties and variability of compressive strength of RAC on an industrial level. The camber deflection of the prestressed beams was measured and correlated with the measured properties of the concrete.
- The last stage of this research included an in-depth analysis of the experimental data. The results were compared with conclusions from previous relevant studies and analytical predictions from different codes and standards. If necessary, new or adapted prediction models were developed to describe the behaviour of RAC accurately. Due to a limited period, only a small number of full-scale beams were tested.

In this doctoral thesis, two frequently used industrial mixtures (target strength class C50/60) from a Belgian precast constructor were set as bases for all

experimental tests. The coarse fraction of limestone (6.3-14 mm) was partially or fully replaced by the high-quality RCA of the same fraction to minimise the change in the particle size distribution of the mixture. An industrial recycling plant in Flanders has produced the high-quality RCA. All high-quality RCA used in the tests of this research originate from the same batch. The influences of different batches of high-quality RCA and other RCA types are outside the scope of this work.

Special attention was given to the variation of the compressive strength of concrete with RCA by means of a statistical analysis of large datasets of concrete with RCA (>20 samples/mean value) in Chapter 5. Other datasets performed within this project are, due to practical reasons, too limited (6 samples/mean value) to be subjected to a thorough statistical analysis. In these cases, only the standard deviation is given to evaluate the variation in the results.

1.7 Outline of the thesis

Chapter 1 introduces the main topics of this thesis, with a first approach to the use of RCA in precast structural concrete. The background of this thesis is set clear. The aim of this work and the research objectives are addressed as well as the research methodology to achieve the preset goals.
Chapter 2 includes a look at the detailed and international approach of the current practices on RCA use. The findings and practical recommendations for specifying and using RCA in precast concrete applications are summarised. A summary of international standards concerning the use of RCA is also presented.
Chapter 3 describes the experimental design of the tests. All the aggregate properties and mix designs used in this research are described.
Chapter 4 continues with the first experimental test topic about the early age mechanical properties of RAC cured in different conditions. The test results of the development of compressive strength, tensile strength, and elastic modulus are discussed. The experimental results are compared to models in existing standards.

Chapter 5 describes the results and analysis of the variability of the compressive strength of RAC with different replacement percentages. The compressive strength of all the mixtures is tested on 1 day and 28 days. The influence of the high-quality RCA on the variability of compressive strength of concrete is statistically discussed. The effect of the difference in variability in properties on the design parameters is determined.

The bond behaviour between reinforcement and RAC are discussed in depth in Chapter 6. The effect of the RCA on the bond properties and the bond-slip relation between precast concrete and steel reinforcement are tested and analysed.

Chapter 7 includes an industrial validation on the influence of RCA on the early-age mechanical properties and the variability of the compressive strength of concrete. The findings from Chapter 4 are compared with results from an industrial campaign to examine the strength development of concrete with the high-quality RCA, subjected to the actual curing conditions in the precast concrete plant. The compressive strength variability of the industrial mixes is tested on 28 days and compared with the conclusions from Chapter 5.

Chapter 8 describes the full-scale beam test on an industrial scale. The influence of RCA on the measured camber after the prestress transfer is discussed. The experimental results are compared with theoretical prediction models of camber on prestressed beams. Adapted parameters to include the effect of RCA in the prediction models are proposed.

Chapter 9 sums up the overall conclusions by addressing the research objectives and questions and future research recommendations.

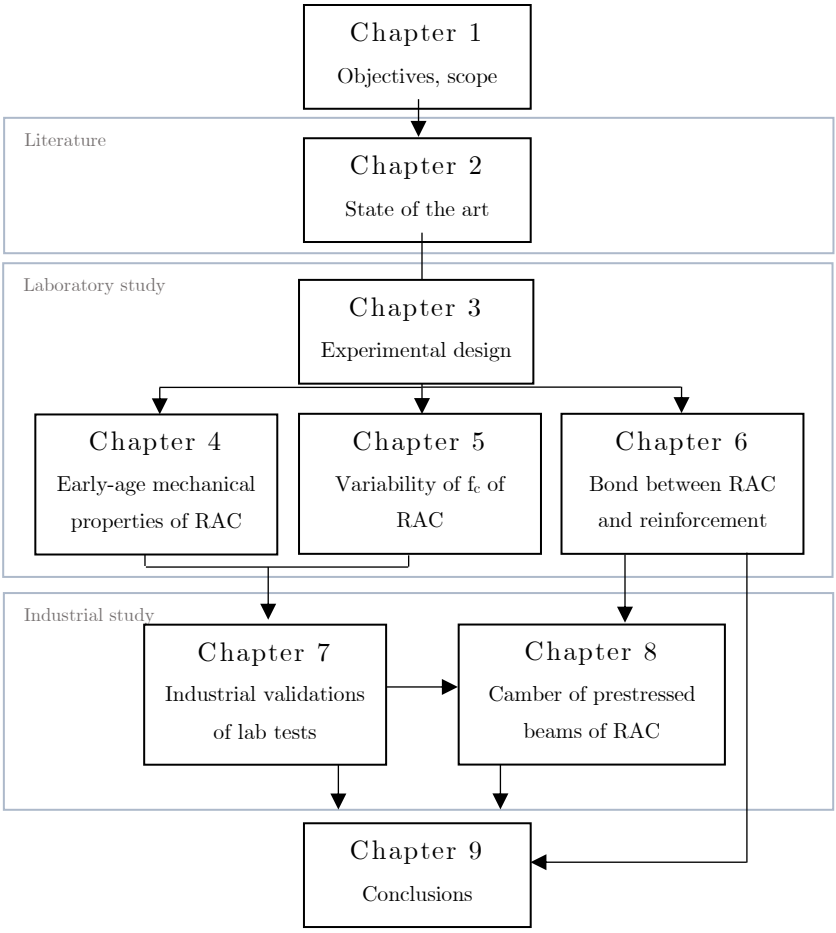


Figure 1-4 – Chapters of the thesis

Chapter II

General background

2.1 Recycled concrete aggregates

Recycled concrete aggregates (RCA) are commonly used in non-structural applications. They serve as a replacement for natural aggregates in bound and unbound applications. A brief overview of the current state-of-the-art, i.e., production, standardisation, and properties, is discussed in this section.

2.1.1 Production of recycled concrete aggregates

The quality of the RCA is largely affected by the production process. Road works used to be the primary RCA source, but lately, demolition works from buildings are increasing. Since the purity of construction and demolition waste streams can be questionable due to the high heterogeneity, it is necessary to optimise all RCA production steps. The increase in demolition works forces the construction industry to search for new solutions for different types of RCA. New solutions should enable the construction industry to use RCA in high-valued applications to generate an economic benefit. Different waste streams can be transformed into high-quality RCA for use in suitable applications.

However, dealing with RCA should be done with care for quality. Many problems can arise when RCA from unknown origins contain unknown contaminants. Strict regulations and attention for every step in the production process of RCA are crucial for advanced use as construction materials. The public opinion on the use of RCA is generally reserved due to the assumed higher heterogeneity. However, selective demolition protocols, effective sorting protocols and efficient recycling processes can limit the heterogeneity and

increase the potential of RCA for use in high-quality construction products. The following subsections describe every step in the production of RCA as being executed nowadays in Flanders, Belgium.

2.1.1.1 Demolition process

Road works are the primary source of high-quality RCA, but their use is usually limited to low-value applications such as road foundations or sub foundations. To upgrade possible applications of this type of aggregates, it is essential to separate different waste streams on the demolition sites. Concrete, asphalt, and linear elements should be separated to create pure waste streams. Top layers of concrete from road demolition form one of the most ideal sources for high-quality RCA due to the concrete's high purity and quality. However, de-icing salt inside the concrete debris can limit possible applications for this waste stream. Lean concrete and concrete mixed with asphalt or soil should be treated differently, as the quality and purity are lower.

RCA from construction and demolition waste (CDW) are generally a mix of concrete with bricks, wood, plastic... Selective demolition is necessary to create relatively pure waste streams. The quality of the incoming CDW in the recycling company will increase, which is beneficial for the final product's quality. This selective demolition starts with a pre-demolition audit, an activity before the demolition process for setting up an inventory of materials and components in the buildings and improve the traceability of different waste streams [41]. The pre-demolition audit consists of an identification of materials and components that will be generated during the demolition, and additional information about which materials should be separated, reused or recycled. The audit can be compulsory or voluntary. A pre-demolition audit is beneficial for the client since the risk of unexpected costs decreases thanks to a thorough inventory, and the rubble can be processed at a lower price. The contractor has less chance for unfair competition and gains an advantage by limiting environmental and health risks. The final benefit belongs to the recycling company, taking advantage of clean waste streams to ensure the purity and quality of different types of recycled aggregates.

Concrete elements and natural stone are the most optimal sources for high-quality RCA. Foundations, concrete blocks, and structural concrete can also fit for high-quality or normal-quality RCA, as long as they can be adequately separated from the rest of the waste streams. Bricks, cellular concrete, insulation, and wooden elements should be avoided to be mixed with concrete and separated into different waste streams.

2.1.1.2 Recycling process

The second key chain in the production of RCA is the recycling process. This starts with the incoming concrete rubble from demolition and road works to the recycling plant. In Flanders, regulations prescribe a first sorting selection based on the origin of the rubble. A distinction must be made between a High Environmental Risk Profile and a Low Environmental Risk Profile. Waste materials from non-selective demolition or public recycling centres belong to the former profile. This type of waste needs to be sorted and treated differently with higher quality control. The treatment is more intensive and expensive. Most recycling plants charge additional costs for this type of waste.

Strict control on the incoming waste is necessary to sort the rubble properly. Concrete rubble from high-quality concrete elements should be separated from low-strength concrete rubble. Bricks, soil, and other waste streams, which negatively influence the final RCA quality, should be separated from the beginning, and processed differently.

Before the recycling process, concrete should be separated from the reinforcement. During the recycling process, large impurities should be removed through hand-picking. Light particles like paper, wood and insulation materials can be removed with a wind shifter, while all ferromagnetic materials are removed using an electromagnet. Non-ferromagnetic materials should be removed by handpicking since they can easily be distinguished visually due to their different surface. Any other impurities can be removed by the sink/float principle, where some materials float or sink in water, making it easier to separate different materials from concrete rubble.

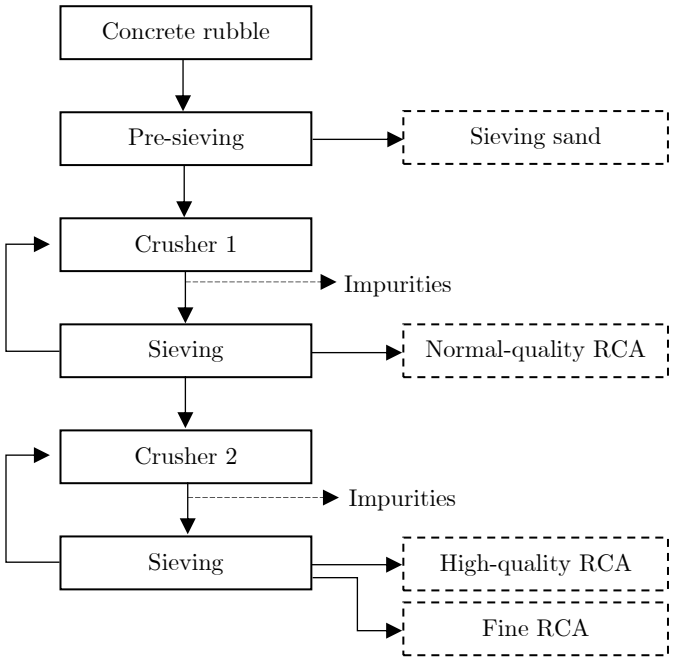


Figure 2-1 – Simplified two-step crushing process

After pre-sieving the concrete debris, the sorted and pure concrete rubble is subjected to a crushing process on a recycling plant. Crushing installations can be divided into fixed and mobile crushers. Large recycling companies usually have a fixed installation with more capabilities for separating impurities. Companies where the recycling activities are secondary (e.g., road construction companies) to other construction works might prefer a mobile installation because of its flexibility. In situations where the concrete rubble can be directly reused, e.g., large infrastructure works might also prefer a mobile crusher. Jaw crushers, cone crushers and impact crushers are the most commonly used crushers in Flanders, Belgium. Each type has its characteristics, advantages, and disadvantages, and can be used to produce RCA. Another essential parameter is the number of crushing steps in the recycling process. Most recycling companies opt to have a one-step crushing process, where the primary crusher is used to process the concrete rubble into RCA. Nevertheless, some advanced companies choose to have a two-step crushing process, where a

secondary crusher is added to the recycling process. In most cases, a two-step crushing process deals with a jaw crusher, followed by an impact crusher. In the case of concrete rubble, the additional second step enables the recycling company to decrease the amount of mortar attached to the RCA. However, the ability to use two crushers increases the operating and manufacturing costs of the recycling installation. A simplified recycling process with a two-step crushing process is shown in Figure 2-1.

After crushing into RCA, a sieving installation divides the crushed aggregates into different fractions. The size of the different fractions depends on the most frequently used applications for recycled aggregates. Fractions of 0-40 mm and 0-20 mm are most commonly produced by the crusher, sometimes divided into a fine fraction, e.g., 0-4 mm and a coarse fraction, e.g., 4-20 mm. The coarse fraction is mainly used for foundation purposes in road works. Other applications for RCA may require different fraction sizes.

2.1.1.3 Quality control of recycled aggregates

Proper quality control of the RCA is necessary since the use of RCA has gradually been standardised in the last decades. EN 206 [42] and NBN B 15-001 [14] describe different requirements that recycled aggregates (RA) should fulfil to be categorised according to their quality and composition. Some of the necessary tests for RCA are:

- *Composition*
  The amount of concrete, natural stone and red bricks in RA is important to define its quality. More concrete and natural stones, and fewer bricks increase the RCA quality. Considerable variation in the number of components affects the properties of the combined material. Therefore, strict regulations are set up to limit the possible heterogeneity of the materials, as shown in the following section.

- *Water absorption, density, and LA-value*
  The water absorption, density and LA-value of the RA are the most commonly used parameters to quantify the quality of the aggregates.

Higher water absorption is caused by a higher amount of attached mortar in RCA. This corresponds with a lower oven-dried density. The Los Angeles abrasion value determines the ability of aggregates to withstand abrasion. A lower value indicates better quality.

Rather than relying on a characterisation based on composition only, many researchers have focused on using performance-based approaches to classify RA. Using quality measures like WA_{24}, density, or LA-value provide a more comprehensive approach to estimate the performance of RA [43].

- *Size distribution*
  The size distribution is an essential parameter in the recycling process. Fine particles (<0.063mm) generally decrease the quality of RA. An improper recycling process produces too many fines with a lower economic value than the coarse fraction.

2.1.2 Standardisation

The use of coarse RA in different applications requires a proper standardisation system to categorise these materials. Dividing the aggregates into different categories based on the quality and/or origin is the only way to create a certain amount of uniformity into recycled aggregates.

2.1.2.1 EN 206

European standard EN 206 [42] is set up to define concrete for structures cast in situ, precast structures, and structural precast products for buildings and civil engineering structures. Following recommendations are set up for the use of coarse RA. Two types of coarse RA with the following properties are described:

- Recycled aggregates type A
  - Rc_{90}, Rcu_{95}, Rb_{10-}, Ra_{1-}, FL_{2-}, XRg_{1-}
  - Minimal oven-dried particle density (ρ_{RD}): 2100 kg/m^3
- Recycled aggregates type B

- Rc_{50}, Rcu_{70}, Rb_{30-}, Ra_{5-}, FL_{2-}, XRg_{2-}
- Minimal oven-dried particle density (ρ_{RD}): 1700 kg/m^3

Type A RA resembles the aforementioned RCA. Type A RA from a known origin can be used in applications for an exposure class similar to that of the original concrete, but only with a maximum replacement percentage of 30%. Type B cannot be used in concrete with a strength class higher than C30/37. The maximum strength class of Type A is not specified in EN 206. The use of the two types of RA in different exposure classes is defined in Table 2-1.

Table 2-1 – Maximal use of recycled aggregates in different exposure classes according to EN 206 (% by mass) [42]

	X0	XC1, XC2	XC3, XC4, XF1, XA1, XD1	Other classes
RCA Type A	50%	30%	30%	0%
RMA Type B	50%	20%	0%	0%

2.1.2.2 NBN B 15-001

Belgian standard NBN B 15-001 [14] is a national annex to EN 206 [42]. NBN B 15-001 and EN 206 are firmly connected and cannot be interpreted independently. NBN B 15-001 also describes two types of recycled aggregates with accompanying demands, similar to EN 206. The requirements for the two types of recycled aggregates are stricter in the national standard NBN B 15-001 compared to EN 206:

- Recycled Concrete Aggregate type A+
  - d≥4mm and D≥10mm
  - Composition category Rc_{90}, Rcu_{95}, Ra_{1-}, $XRg_{0.5-}$ and FL_{2-} [21]
  - Minimal categories FI_{20}, $f_{1.5}$, LA_{35}, $SS_{0.2}$, A_{40} [21]
  - Minimal oven-dried density (ρ_{RD}) of 2200 kg/m^3
  - Maximal water absorption of 10%
- Mixed Recycled Aggregate type B+
  - d≥4mm and D≥10mm

- Composition category Rc_{50}, Rcu_{70}, Rb_{30-}, Ra_{5-}, XRg_{2-} and FL_{2-} [21]
- Minimal categories FI_{50}, LA_{50}, $SS_{0.2}$, A_{40} [21]
- Minimal oven-dried density (ρ_{RD}) of 1700 kg/m^3
- Maximal water absorption of 15%

RCA type A+ can be used for concrete with a maximal strength class of C30/37, while Mixed Recycled Aggregate type B+ can only be used in concrete with a strength class of maximal C20/25. The use in different exposure classes is defined in Table 2-2. Environmental classes are defined in NBN B 15-001 as an extension to the exposure classes defined in EN 206, because of their frequent use in Belgium. Therefore, these environmental classes should be considered in the design of concrete in Belgium. The use of RA in different environmental classes are therefore also specified in Table 2-3 and Table 2-4. Any use of RA in higher strength classes or replacement percentages in specific applications should be tested on feasibility.

Table 2-2 – Maximal use of RA in different exposure classes according to B15-001 (% by volume) [14]

	X0	XC1	XC2	XC3, XC4, XF1, XF3, XA1, XD1	Other exposure classes
RCA type A+	50%	30%	30%	20%	0%
RMA type B+	20%	20%	0%	0%	0%

Table 2-3 – Maximal use of RA in different environmental classes for plain concrete (% by volume) [14]

	E0, EI, EE1	EE2, EE3, ES1, ES2, ES3, EA1	EE4, ES4, EA2, EA3
RCA type A+	50%	20%	0%
RMA type B+	20%	0%	0%

Table 2-4 – Maximal use of RA in different environmental classes for reinforced concrete (% by volume) [14]

	EI	EE1	EE2, EE3, EA1	EE4, ES1, ES2, ES3, ES4, EA2, EA3
RCA type A+	30%	30%	20%	0%
RMA type B+	20%	0%	0%	0%

The maximal allowed replacement percentage in similar environmental classes in NBN B 15-001 is lower than EN 206. In addition, the maximal allowed strength class for the use of type A+ and B+ are lower for the national standard B 15-001. These stricter regulations indicate a more conservative approach of the Belgian standardisation committee towards using recycled aggregates in concrete.

2.1.2.3 EN 13369

The use of RCA in precast concrete is specified in EN 13369 [34]. In general, it is specified that the replacement in weight up to 5% of the total aggregate skeleton is allowed. However, if the RCA originate from precast elements made in the precast plant itself, it is allowed to use up to 10% of the weight of the total aggregate skeleton. This amount can be increased up to 20%$_{M/M}$ in the following conditions:

- The mechanical strength of the concrete product is determined by calculation, possibly backed up with full-scale tests. All the relevant properties of the concrete should be determined with specific tests.
- The mechanical strength of the concrete products is tested on a real scale.

A replacement percentage higher than 20%$_{M/M}$ can be used when all the design properties of the concrete are tested. The calculated properties should be compared with initial full-scale tests. The RCA should be used in concrete with higher durability requirements than the concrete where the RCA originated. Other RCA, delivered from a recycling plant, can be used up to half of the

percentage allowed for aggregates from precast elements made in the precast plant itself.

2.1.3 Properties of recycled concrete aggregates

The properties of RCA are strongly dependent on the origin of the concrete rubble and the recycling process. Different qualities of concrete rubble result in different properties of RCA. A recent Flemish industrial research project [22] concerning RCA in concrete determined that the properties of RCA depend on the different concrete waste streams. The type of crusher has a considerable influence on the shape and size distribution of the RCA, but the WA_{24} and density of the RCA do not change significantly [22]. It is theoretically optimal to store the different concrete waste streams into different stockpiles on the recycling plant. However, most recycling companies cannot store every concrete waste stream separately. Nevertheless, if possible, it can be beneficial to upgrade the use of RCA by creating two different waste streams:

- Concrete rubble 1: precast concrete, the top layer of road construction, natural stone → waste stream to produce **high-quality RCA.**
- Concrete rubble 2: all other types of concrete rubble → waste streams to produce **normal-quality RCA** – similar to type A according to [42] and type A+ according to [14].

In most cases, the high-quality RCA are not directly sold on a recycling plant. If this type is produced, it is mainly used to replace natural aggregates in ready-mixed concrete production at the recycling plant. However, most recycling plants do not receive this waste stream because of limited connections to road construction companies, or they cannot stockpile it independently.

The properties of RCA originating from different types of concrete rubble are discussed below.

Concrete from top layers in road construction

Using concrete rubble from top layers of road construction is one of the best sources to produce high-quality RCA in a recycling plant. A WA_{24} of 3.5% and a LA value of 30 are measured by Noterman et al. [22]. Novakova and Mikulica

[44] used demolished concrete road surface of 1.5 years old with a strength class of C30/37 as a base for RCA. The WA_{24} of the fraction 8-16 mm is 6.3%.

Precast concrete

The waste stream from precast concrete can be separated into two fractions [22]. Production residues and production debris form the first group of precast concrete; a second group consists of old precast products derived from the demolition of industrial buildings. The RCA quality derived from the two groups does not differ significantly if appropriately sorted, but the risk of contamination with impurities is higher in the second group.

The tests from Noterman et al. [22] show a good quality of RCA produced from precast concrete: $WA_{24} = 4.50\%$, LA = 35. Brandes and Kurama [38] also tested RCA from rejected precast concrete, resulting in a WA_{24} of 3.71%. Siempu and Pancharthi [45] used RCA from recycling precast concrete members with a WA_{24} of 3,0%. Rahal [18] used 20 – 40 years old precast concrete beams to produce high-quality RCA with a WA_{24} of 3.47%. Gonzalez and Etxeberria [46] used precast concrete of different strength classes as bases for RCA production. Precast concrete of 40, 60 and 100 MPa were processed into RCA with a WA_{24} of respectively 5.91%, 4.90% and 3.74%, confirming the principle of a lower WA_{24} with higher strength parent concrete.

Ready-mixed concrete

Tests on RCA produced from ready-mixed concrete show WA_{24} of 5.50% and LA40 [22], but it seems that the variability between these properties can be more considerable compared to the first two groups. The variation in conditions at the moment of concrete pouring and the variation in ready-mixed concrete quality can explain this. Seara-Paz et al. [47] tested RCA from ready-mixed concrete from structural elements and concluded a WA_{24} of 5.4%. Butler et al. [48] used RCA from the demolition of an airport terminal, resulting in a WA_{24} of 6.15%. At present, in Flanders, no distinction is made between precast concrete and ready mixed concrete in the demolition process [22], although a more thorough distinction based on quality can be beneficial for possible applications of RCA.

Natural stone elements

The use of natural stone elements for RCA production yields a good quality of aggregates, as expected (LA30, $WA_{24} = 1\%$). Also, in this category, a difference can be made between residues from the production of natural stone elements and the demolition of constructions with elements made of natural stones. The limited availability complicates the ability to keep this waste stream uncontaminated.

Concrete foundation and small concrete elements

Foundations layers of concrete and small concrete elements can be used as an RCA source. The recycling of small concrete elements can result in good quality RCA. However, the composition of these waste streams can be variable, which complicates producing RCA with uniform quality. Butler et al. [48] tested the WA_{24} of crushed small concrete road elements and concluded a value of 4.66%.

2.2 Precast industry

2.2.1 General

A precast concrete product is a concrete element produced in a factory before being transferred to the construction site, to be assembled with other elements in a larger structure. These elements are prepared, cast, and hardened in a specially equipped and controlled environment in a permanent location. This type of concrete production has different advantages. Concrete products can be produced independently of different weather conditions, limiting the concrete production at the construction site. The controlled environment will decrease any interruptions of the concrete production process and enhance the final element's quality. The concrete properties of the produced elements can easily be tested and controlled before placing the elements on the construction site. This enhances the concrete quality control.

The use of RCA in precast concrete will affect different steps in the production process of a precast plant. The following sections describe the current situation

concerning RCA in the precast concrete and the possible future impacts for the precast sector.

2.2.2 State of the art

Currently, the precast industry in Flanders, Belgium, already uses limited amounts of recycled aggregates in their production process. Production leftovers of fresh concrete are washed out to remove the cement particles and keep the natural aggregates in their original state. The natural sand and coarse natural aggregates are reused into new precast concrete products. The cement slurry is put aside, dried, and mainly disposed as waste. Some preliminary research projects have been trying to determine ways to recycle this product [49]. Hardened production residues or rejected concrete products are sometimes crushed and used as RCA into precast concrete. This waste stream has a high potential, as discussed in 2.1.1.2, but in most cases, the hardened concrete on a precast plant is transported to a recycling plant and used in other applications.

2.2.3 Interest in using RCA

The precast industry in Flanders has shown recent interest in using more RCA in their precast concrete products. Different companies are interested in developing an innovative business model to close the material circle in precast concrete [22]. The demolition of high-strength concrete elements gives opportunities to recycle demolished concrete into a full-fledged primary material, which can be used in similar applications as the original application. This transition will improve material management in the precast industry, resulting in sustainable concrete production.

2.2.3.1 Future perspectives

From a practical point of view, there are two possible ways to increase RCA use in the precast industry.

Circular use of recycled concrete aggregates in a local precast plant

As mentioned before, recycling precast leftovers and rejected products into RCA at a local precast plant have a high potential. The recycling of this particular waste stream has the advantage of a known origin and a high level of purity. In this case, a mobile crusher should be available to process the concrete leftovers at regular time intervals. This is an expensive process that can become economically profitable only if the percentage of leftovers and production debris is high enough so that the economic benefit of the decreased use of natural aggregates can cover the cost of the mobile crusher. Precast companies that are part of a larger construction holding might be able to organise a transfer system for the mobile crusher between different precast companies of the same holding.

Use of high-quality RCA from a recycling plant

Using high-quality RCA bought from a recycling company to produce precast concrete requires specific quality control. RCA can be used in precast concrete only if it can reach a minimum quality requirement specified in standards. These quality limits can vary according to the use in different precast products. Prestressed and architectural concrete requires a higher quality of aggregates than normal reinforced and industrial concrete. A high amount of concrete particles in RCA, a low amount of floating particles, a low amount of fines and a low LA-value are required to use RCA in precast structural concrete. Further sorting of concrete rubble is necessary to ensure a high quality of RCA. All pure precast production debris, top layers of roads, and natural stones should be sorted separately and used as a high-quality RCA source. Pure, high-quality RCA with a maximal WA_{24} of 4-5% should be able to be produced with an optimal source of concrete rubble, combined with an optimal recycling process with two crushers and abilities to remove floating particles and fines.

2.2.3.2 Impact for the sector

The most apparent impact on the concrete sector is the environmental advantage. The use of RCA will decrease the need for primary raw materials. Most primary aggregates for use in concrete in Flanders need to be imported from neighbouring regions and countries, while RCA can be locally produced in more than 200 recycling companies in Flanders. Transportation distances will

decrease, resulting in less CO_2 emissions and transportation costs. It is estimated that the yearly certified production of CDW in Flanders will increase to 15M tons by 2025. 3 to 4M tons of this CDW may potentially be fit for use in high-end applications such as precast structural concrete [22].

The recycling of RCA, originating from high-strength concrete, into precast concrete products demands some changes in the production process of precast concrete, as discussed before. The different properties of RCA compared to natural aggregates demand a thorough quality control on material and product level. The production staff should be experienced in this matter to fully understand and correctly apply the use of RCA into precast concrete products.

Economic impact of using RCA in precast concrete

The economic aspect of the use of RCA in precast concrete is crucial in this matter. Noterman et al. [22] have set up a case study to evaluate the economic impact of using RCA in precast elements. The basis for this case study is a hypothetical demolition of an industrial building where 10 000 tons of concrete is being recycled. The building contains 3000 tons of precast elements where the concrete is of high strength and high purity. 2000 tons of concrete floors are present with proper strength and a proper purity level. The final 5000 tons of ready-mixed concrete can be mixed with impurities from other waste materials. Average market prices have been used to calculate the cost of different scenarios for the demolition process, recycling process and replacement percentage of RCA in precast concrete.

- Demolition process

  The demolition process, as if organised nowadays, where the building is demolished with the separation of different waste streams, will result in 10 000 tons of removed concrete, all in the same batch. A more thorough demolition process can result in a separation of the different concrete debris. This method will result in extra costs for an extra inventory and extra-human resources but will be economically beneficial since the acceptance cost for the 3000 tons of precast concrete and 2 000 tons of concrete floors will be less than a mixed

acceptance cost. This results from high demand at recycling plants for a high quality of concrete rubble. The complete calculation reveals that the highly selective demolition process results in a lower cost (-3%) than the standard selective demolition process [22].

- Recycling process

  There are different possible scenarios for handling concrete rubble in the recycling process. The standard recycling process where all the concrete rubble is handled with the same method acts as a reference with an outcome of RCA 4-40 mm and fine RCA 0-4 mm. Technical research should determine whether two crushers or one crusher are sufficient to create high-quality RCA. Extra recycling steps in the recycling process will increase RCA quality and increase the operating cost. The cost of using a second recycling step in the process can be compensated by a higher economic value of the high-quality RCA. The profit made for the recycling company turned out to be in both ways similar to the reference recycling process. The economic feasibility will be dependent on the market price of high-quality RCA and the price of natural aggregates.

- Precast concrete production process

  In this case, the reference production process is a scenario where only natural aggregates are used. The cost of this process is compared with a process where a low replacement percentage (10%) of natural aggregates by RCA is taken. A third scenario represents a case where a higher replacement rate (30%) is used. The RCA cost can be close to the cost of natural aggregates in a pessimistic assumption, mainly because of extra costs for quality control, adaptions of mixtures... A higher replacement percentage still turns out to be more cost-effective than a lower replacement percentage or the use of only natural aggregates. The decrease in transportation costs mainly makes the difference in this case.

All these different scenarios can be combined to determine the profit made by the different parties. The most optimal situation for all parties is a selective

demolition, a one-step recycling process (on the condition that this is technically possible) and a 30% replacement of the natural aggregates by high-quality RCA in the precast elements. Since the transportation cost is one of the most critical factors in this process, estimations have shown that an average replacement percentage of 20% can result in an economisation of 1.74M euro for a market demand of 600 000 tons of aggregates [22].

From the above simulation, an economic benefit for different actors producing precast concrete with RCA appears feasible. Actors in the demolition process will benefit from a lower acceptance cost for concrete rubble, actors in the recycling process will benefit from a higher economic value of RCA and actors in the precast industry will economically benefit when higher replacement percentages are used. This simulation is, of course, simplified to estimate the feasibility. In practice, several other factors will come into play to influence the economic benefit for different actors.

Chapter III

Experimental design

Two industrial mixtures are used as a start in this research. One is a conventional concrete mixture; the other is self-compacting concrete. The used materials and the mix proportions of the concrete mixtures are given in this chapter.

3.1 Materials

3.1.1 Cement

The cement used in the present study is CEM I 52.5 R HES conform to EN 197-1 [50], which is a rapid hardening Portland cement with high-early strength (HES). The cement's chemical composition is shown in Table 3-1. The absolute density is 3150 kg/m^3; the Blaine fineness is 489 m^2/kg. This type of cement is frequently used in the precast concrete industry to achieve high-early strength for concrete.

Table 3-1 – Chemical composition of cement

CaO	SiO_2	Al_2O_3	Fe_2O_3	MgO	Na_2O	K_2O	SO_3	Cl-
62,8%	18,7%	5,7%	4,2%	1,0%	0,44%	0,63%	3,7%	0,06%

3.1.2 Limestone powder

Limestone powder Calcitec 2001M is used as a filler in self-compacting concrete (SCC). This type of limestone powder is a stable and economical filler with an average calcium carbonate content of 98.8%. The absolute density is 2770 kg/m^3, the Blaine and BET fineness are 486 m^2/kg and 918 m^2/kg. The chemical composition and particle size distribution of the limestone powder are shown in Table 3-2 and Figure 3-1, respectively.

Table 3-2 – Chemical composition of limestone powder in $\%_{M/M}$

$CaCO_3$	SiO_2	Al_2O_3	Fe_2O_3	MgO
98.8	0.18	0.03	0.04	0.32

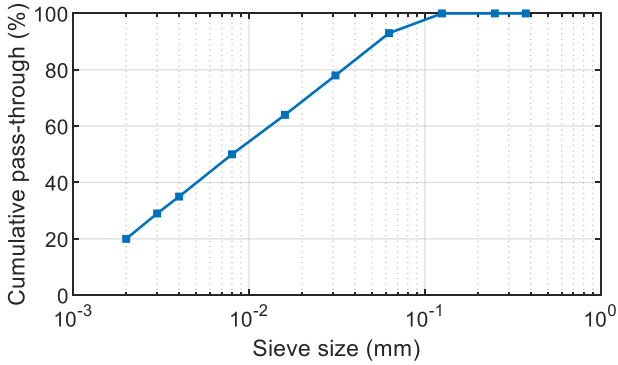


Figure 3-1 – Particle size distribution of limestone powder

3.1.3 Aggregates

3.1.3.1 Natural aggregates

The natural aggregates include washed sea sand 0-2 mm, limestone 2-6.3 mm and 6.3-14 mm. They are shown in Figure 3-2 (a-c). Figure 3-3 presents the particle size distributions of the aggregates determined according to NBN EN

933-1 [51]. The particle densities, including the apparent density (ρ_a), oven-dry density (ρ_{rd}), saturated surface-dry density (ρ_{ssd}) and the 24h water absorption (WA_{24}) of the aggregates are measured according to NBN EN 1097-6 [52] and presented in Table 3-3.



(a) Washed sea sand 0-2 mm



(b) Limestone 2-6.3 mm



(c) Limestone 6.3-14 mm



(d) RCA 6.3-14 mm

Figure 3-2 – Aggregates used in this study

Table 3-3 – Properties of aggregates used in this study

Aggregate	ρ_a (kg/m^3)	ρ_{rd} (kg/m^3)	ρ_{ssd} (kg/m^3)	WA_{24} (%)
Sand 0-2 mm	2610	2580	2590	0.40
Limestone 2- 6 mm	2710	2680	2690	0.45
Limestone 6.3-14 mm	2700	2650	2670	0.78
RCA 6.3-14 mm	2660	2380	2490	4.46

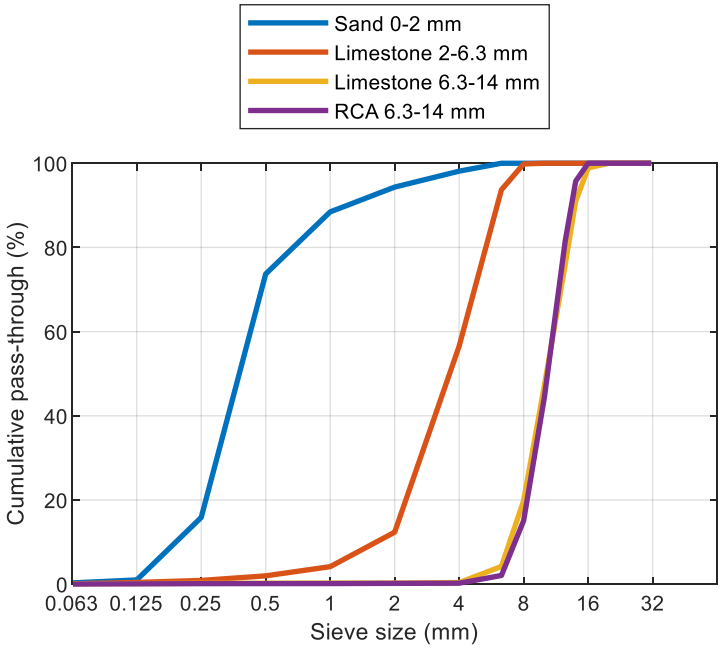


Figure 3-3 - Grading curves of the used aggregates determined according to NBN EN 933-1 [51]

3.1.3.2 Recycled concrete aggregates

The high-quality RCA, with a fraction of 6.3-14 mm, are supplied by a recycling plant in Flanders, Belgium. The RCA origin from pure concrete waste from pure ready-mixed concrete and precast production leftovers. An optimised two-step recycling process with a jaw crusher and an impact crusher was used to produce high-quality RCA from the concrete waste. The fraction 6.3-20 mm was obtained from the industrial crusher and has been sieved in the laboratory of KU Leuven Campus Bruges on 14 mm to create the fraction 6.3-14 mm. The high-quality RCA are used to partially or entirely substitute the natural aggregates of the same fraction, i.e., limestone 6.3-14 mm. The high-quality RCA are shown in Figure 3-2 (d). Figure 3-3 shows that the grading curve of the RCA is close to that of the limestone 6.3-14 mm, indicating that the

influence due to the difference in the particle size distribution of the aggregates is minimised.

The particle densities and the 24h water absorption (WA_{24}) of the RCA is measured according to NBN EN 1097-6 [52] and presented in Table 3-3. Based on the measured properties, it can be concluded that the RCA satisfies the acceptance criteria for RCA of Type A+ prescribed in NBN B 15-001 [14] ($\rho_{rd} \geq 2200$ kg/m^3 , $WA_{24} \leq 10$ %). They can be classified as Type II RCA ($\rho_{rd} \geq 2000$ kg/m^3 , $WA_{24} \leq 10$%, allowable concrete strength class $\leq$ C50/60) in the RILEM recommendation [53] and as Class M RCA ($\rho_{rd} \geq 2300$kg/m^3 , $WA_{24} \leq 5$%) according to JIS A 5022 [54].

3.1.4 Water

The used water for mixing concrete is regular tap water at room temperature in the laboratory of KU Leuven campus Bruges.

3.1.5 Admixtures

3.1.5.1 MasterGlenium 51 con 35%

MasterGlenium 51 con 35% is a superplasticizer based on a chemical chain of modified polycarboxylic-ether, developed for precast concrete with high durability standards, concrete quality, high-early strength, and aesthetic surfaces. It is used to improve the workability of concrete mixtures with low W/C ratios and high (early) strength.

3.1.5.2 MasterEase 5035

MasterEase 5035 is a superplasticizer based on polyarylether. This admixture is developed for use in precast concrete and prestressed concrete mixtures. It can be easily applied in precast self-compacting concrete as it vastly improves the workability without inducing interference in the concrete mixture.

3.2 Mix design

Two industrially designed mixtures are used during this research as reference mixtures.

3.2.1 Conventional concrete C50/60

This mixture is directly adopted from an industrial concrete plant where the mixture is used to produce prestressed concrete beams with a **variable height**, as shown in Figure 3-4. The prestressed beams are mainly used in the structural skeleton in industrial buildings. This mixture is designed to create a high-early strength, necessary to decrease the demoulding time and the moment when the prestress releases. The mixture is designed to reach a minimal strength class of C50/60.



Figure 3-4 – Prestressed beams with a variable height [55]

In the following chapters, five different mixtures of this precast structural concrete are designed. The difference between the concrete mixtures is the replacement percentage of limestone 6.3-14 mm by the RCA, varying from 0, 10, 20, 50 to 100%$_{V/V}$. The different mix proportions are shown in Table 3-4. The concrete mixtures are named RAC, followed by a number indicating the replacement percentage of limestone 6.3-14 mm by the RCA 6.3-14 mm. For instance, RAC50 refers to a concrete mixture with 50 %$_{V/V}$ of high-quality RCA.

The mixture RAC0 refers to a concrete mixture without RCA, which serves as a reference concrete.

In the present work, the RCA partially or entirely substitutes the 6.3-14 mm fraction of the coarse natural aggregates. When the total coarse aggregates (≥ 4mm) is taken into account, the RCA volume replacement percentage of these five mixtures becomes 0, 9, 18, 45 and 90%$_{V/V,}$ respectively. When the complete aggregate skeleton is considered, the RCA replacement percentages become 0, 4, 9, 22 and 45%$_{V/V}$.

Table 3-4 – RAC concrete mix proportions

	RAC0	RAC10	RAC20	RAC50	RAC100
CEM I 52.5 R HES (kg/m^3)	400	400	400	400	400
Free water (kg/m^3)	192	192	192	192	192
Effective W/C ratio	0.48	0.48	0.48	0.48	0.48
Absorption water (kg/m^3)	7	10	13	22	38
Sand 0-2 mm (kg/m^3)	780	780	780	780	780
Limestone 2-6.3 mm (kg/m^3)	196	196	196	196	196
Limestone 6.3-14 mm (kg/m^3)	785	706	628	392	0
RCA 6.3-14 mm (kg/m^3)	0	74	148	371	742
Superplasticizer [kg/m^3)	1.3	1.3	1.3	1.3	1.3
Slump value (mm)	185	190	185	190	190

The water absorption of the aggregates needs to be considered when mixing concrete. When the moisture content of the aggregates is below the SSD (saturated and surface dry) state, extra absorption water needs to be added to the mixture to reach the designed effective water-to-cement ratio (W/C$_{eff}$). This is mostly the case when RCA are used in concrete mixtures. Due to the high water absorption of RCA, it is essential to accurately determine the moisture content and the water absorption of the RCA. An overestimation or underestimation of the WA_{24} affects the mechanical properties and the workability [56]. In Table 3-4, the absorption water is calculated based on the mass of the aggregates, under the assumption that all aggregates are oven-dry.

The workability of the mixtures is tested several times by measuring the slump value according to NBN EN 12350-2 [57]; the average value of the tests is shown in Table 3-4. The designed workability class S4 is reached for all mixtures due to the addition of the compensation water and superplasticizer of the type MasterGlenium 51 con 35%.

3.2.2 Self-compacting concrete C50/60

This precast self-compacting concrete mixture is directly adopted from an industrial concrete plant where the mixture is used in manufacturing prestressed concrete beams with a **constant height**, as shown in Figure 3-5. This mixture is designed, similar to the previous mixture, to create a high-early strength, necessary to decrease the demoulding time and moment of the prestress force release. The mixture is designed to reach a minimal strength class of C50/60.



Figure 3-5 – Prestressed beam with a constant height [58]

Three different concrete mixtures are designed and produced. The difference between the concrete mixtures is the partial or total replacement of the limestone 6.3-14 mm by the RCA 6.3-14 mm, which varies from 0, 20 and 100 $\%_{V/V}$. The mix proportions of the mixtures are shown in Table 3-5. The concrete mixtures are named RASCC, followed by a number that indicates the replacement percentage of the limestone 6.3-14 mm by the RCA 6.3-14 mm. For instance, RASCC20 refers to a concrete mixture with $20\%_{V/V}$ replacement. The

mixture RASCC0 refers to a concrete mixture without any RCA, which serves as a reference concrete.

In the present work, the RCA partially or entirely substitutes the 6.3-14 mm fraction of the coarse natural aggregate. When the total coarse fraction (≥ 4mm) is taken into account, the RCA replacement percentage becomes 0, 14 and 71%$_{V/V}$, respectively. When the total aggregates skeleton is considered, the RCA replacement percentages become 0, 5 and 27%$_{V/V}$.

Table 3-5 – RASCC concrete mix proportions

	RASCC0	RASCC20	RASCC100
CEM I 52.5 R HES (kg/m^3)	400	400	400
Limestone powder (kg/m^3)	120	120	120
Free water (kg/m^3)	200	200	200
Effective W/C ratio	0.5	0.5	0.5
Absorption water (kg/m^3)	9	11	23
Sand 0-2 mm (kg/m^3)	762	762	762
Limestone 2-6.3 mm (kg/m^3)	425	425	425
Limestone 6.3-14 mm (kg/m^3)	440	352	0
RCA 6.3-14 mm (kg/m^3)	0	79.7	391
Superplasticizer (kg/m^3)	4.5	4.5	4.5

In Table 3-5, the absorption water is calculated identically to Section 3.2.1, based on the mass of the aggregates and under the assumption that all aggregates are oven-dry.

3.3 Mixing procedure

The mixing procedure of the concrete used in the following chapters is specified in the following sections. The experimental campaign is divided into laboratory tests and industrial tests.

3.3.1 Laboratory tests

The preparation of laboratory specimens was done in the Laboratory of Civil Engineering, KU Leuven Campus Bruges. All concrete mixtures were mixed in an 80-litre capacity mixer of 3 kW (see Figure 3-6) according to a prescribed two-step mixing process [59]. The aggregates were added to the mixer with half of the calculated total amount of water and mixed for two minutes. Secondly, the cement was added, followed by two minutes of mixing. After that, the rest of the water mixed with the superplasticizer was added. The whole mixture was then mixed for another two minutes.

The concrete mixtures described in Section 3.2.1 were compacted by a 50-60 Hz vibration table. Both cubes (150× 150× 150 mm) and cylinders (Φ 150× 300 mm) were cast and compacted in three layers with 15 s of vibrations each. The SCC mixtures described in Section 3.2.2 do not require any external compaction energy to reach a desired state of compaction. All the samples were demoulded 20 to 24 h after casting.



Figure 3-6 – Laboratory concrete mixer

3.3.2 Industrial tests

The concrete mixing for industrial tests was done in a Haarup industrial mixer, as shown in Figure 3-7. A two-stage mixing process, similar to the laboratory tests, is used [59]. The aggregates were firstly added to the mixer with half of the water and mixed for one minute. Secondly, the cement and limestone powder were added, followed by two mixing minutes. After that, the rest of the water with a superplasticizer was added. A final two minutes of mixing was done before finishing the mixing process.



Figure 3-7 – Industrial concrete mixer (left) and transport of the concrete (right)

Chapter IV

Early-age mechanical properties of recycled aggregate concrete

The development of mechanical properties of precast concrete is crucial for the production efficiency of precast concrete plants. High pressure on the construction planning of precast concrete elements has led to subjecting them to loads and other influences before a curing age of 28 days. A later demoulding time of certain elements results in a lower production rate since available space in the plant and availability of the moulds are often bottlenecks in the production chain in precast concrete plants.

Early-age mechanical properties of precast concrete influence several stages in its production process. The incorporation of RCA in precast concrete possibly influences the development of mechanical properties. The change in water balance within the concrete due to a higher water absorption of the RCA can affect the strength development of the concrete. To avoid significant changes in production schedules and consequently production costs, a thorough investigation of the effect of RCA on the development of the mechanical properties with time is essential. The compressive strength, tensile splitting strength, and elastic modulus are the main mechanical properties used to design different precast concrete elements. The development of these properties is tested with different replacement percentages of RCA. These three properties are selected based on their importance in designing precast elements. Compressive strength is generally considered to be the most crucial parameter of concrete. The demoulding time, the moment of prestress transfer and practical applications where the concrete is subjected to early loads require a

good understanding of the compressive and tensile strength development. The tensile strength development is directly related to the shear strength and cracking when applying loads on the elements [60]. The static elastic modulus of concrete at early ages is an essential parameter in designing prestressed concrete elements to estimate prestress losses and early-age deformations. The influence of RCA on the development of mechanical properties of the conventional precast mixture as described in 3.2.1, and the SCC precast mixture, as described in 3.2.2, are investigated.

Current models for predicting the early-age behaviour of concrete are based on mechanical properties developed from testing conventional concrete. This causes some concerns that the design of precast structural concrete incorporating high-quality RCA may no longer be accurate when current equations are applied. Different models will be developed based on experimental data to determine how the early-age behaviour of high-strength concrete with high-quality RCA can be predicted.

4.1 Strength and modulus of elasticity of precast structural concrete with high-quality RCA at early ages

4.1.1 Previous studies

4.1.1.1 Influence of RCA on compressive strength development

It is generally known that the use of RCA as a replacement of natural coarse aggregates influences the mechanical properties of concrete, as concluded by different researchers [16]. The RCA consist of natural aggregates and attached mortar. The quality and the amount of the attached mortar determine the influence of RCA on the mechanical properties of concrete. Due to the attached mortar, the density of RCA is lower, while the water absorption is considerably higher than natural aggregates. Therefore, when RCA are incorporated in concrete, extra absorption water needs to be added to compensate for the WA_{24} of the RCA. The change of the water conditions in the concrete possibly

influences the strength development of RAC. Previous studies mainly focussed on the early-age compressive strength of RAC. The early-age period is not explicitly defined in relevant research. The interpretation of this period depends on the investigated characteristics of concrete. The relative development of characteristics, also described by the age factor, will be used to compare the development of specific properties of concrete in the following analysis. The age factor of the compressive strength is defined as the ratio of the mean compressive strength concrete at any time, denoted as $f_{cm}(t)$, to the mean compressive strength at 28 days, denoted as f_{cm}. The age factor can be calculated for both cylinders and cubes.

An overview and details about some previous research can be found in Table 4-1. Several studies on the early-age mechanical properties of RCA have been conducted in the past years [4-15]. Several researchers report a similar tendency of a lower age factor at early ages for RAC than concrete with natural aggregates [4-10]. The relative strength of curing ages up to 7 days to the 28 days compressive strength is repeatedly lower for RAC. This observation suggests that the RCA in the mixture influences the hardening process and that RAC has a slower hydration speed in the first seven days and a higher strength gain between day seven and day 28 after the concrete production. This phenomenon is discussed by several authors and attributed to the specific properties of the RCA. A larger WA_{24} results in extra absorption water in the mixture. The aggregates absorb this water shortly after the mixing procedure. After a while, this water bleeds to the surrounding mortar, influencing the hydration process in the interfacial transition zone [4-6].

Manzi et al. [61] researched the effect of RCA on the properties of concrete, both at an early age and later age. The compressive strength of the reference concrete at 7 and 28 days was compared to that of concrete with fine and coarse RCA, originating from the demolition of a concrete building. The authors concluded that the age factor at seven days is lower for RAC than NAC. This phenomenon is assigned to a later reaction of un-hydrated cement inside the attached mortar of the RCA or to a later absorption of water into the RCA, creating a zone with a lower W/C ratio around the RCA. Eiras-Lopez et al. [62] also noticed a clear difference in strength development of concrete with RCA.

The age factor at three days is 0.64 for RAC, while that of NAC is 0.74. This difference is also noticeable for the age factor at seven days. Corinaldesi and Moriconi [63] reported test results from the compressive strength of RAC at 1, 7 and 28 days. The age factor at one day was remarkably lower for RAC compared to NAC. This effect was not reflected in the age factor at seven days, which was similar for RAC and NAC.

Other factors influencing the strength development of RAC were also investigated in the literature. Both Rahal [18] and Hani et al. [64] tested the influence of different strength classes on the strength development of RAC. Rahal [18] reports a study on the influence of RCA on the mechanical properties of concrete with different strength classes, as mentioned in Table 4-1. The RCA used in this study originated from concrete debris from the demolition of a concrete building crushed by a jaw crusher in the laboratory. The authors concluded that the age factor at seven days is lower for RAC than NAC. This phenomenon becomes more apparent with higher compressive strengths. Hani et al. [64] varied the strength classes of concrete using different W/C factors, i.e., 0.4, 0.5 and 0.6. A considerable difference between the age factor at seven days of RAC and NAC was noticed with the low strength concrete. The strength development difference decreases as the strength classes of the concrete increase.

Kou et al. [17] investigated the effect of different qualities of RCA on the early-age compressive strength of concrete. Three RCA types were used with a water absorption of 6.2, 5.6 and 3.8%. As expected, the compressive strength of RAC decreased when higher replacement percentages of RCA were used. The results showed that the age factor at one and four days was lower for RAC than for NAC. The age factor at seven days was similar for RAC and NAC, except for the concrete with the best quality of RCA. The age factor at seven days of RAC with a water absorption of 3.8% is lower than NAC.

Most studies show that RAC generally has a lower age factor at seven days than NAC. Nevertheless, some researchers have reported test data that showed other tendencies. A preliminary study from Brandes and Kurama [38] on the effect of RCA on the strength development of concrete concluded that the RCA have no considerable difference in the strength development of concrete. Similar

results have been reported by Sim and Park [65], Gonzalez and Etxeberria [46] and Pedro et al. [66]. In contradiction with all studies above, the test results from Olorunsogo [67] and Gao et al. [68] even showed a higher age factor at 3 and 7 days for RAC than NAC.

Based on existing studies in the literature, it is concluded that the early-age compressive strength development changes with the incorporation of RCA into the concrete. As can be seen from the above literature study, existing test results on the effect of RCA on the strength development are not consistent. One would assume that a higher WA_{24} of the RCA would result in a more considerable difference in strength development between RAC and NAC, but the several research projects using different RCA qualities are not in line with this matter. In addition, most studies are focused on the early-age mechanical properties of low-strength RAC, and/or make use of laboratory-produced RCA. Therefore, new industrially oriented research is necessary to understand better the influence of high-quality, industrially produced RCA on the early-age mechanical properties of high-strength precast concrete.

Table 4-1 – Influence of RCA on the strength development of concrete in previous research

	Strength class	Water absorption RCA	Production of RCA	Testing ages	$f_{cm,RAC}(7d)$ /$f_{cm,RAC}$	$f_{cm,NAC}(7d)$ /$f_{cm,NAC}$
Manzi et al. [61]	C30/37	7,00%	Industrial	7-28 days	0.85	0.89
Corinaldesi and Moriconi [63]	C40/50	7.50%	Industrial	1-7-28 days	0.82	0.83
Eiras-Lopez et al. [62]	C40/50	5.40%	Industrial	3-7-28 days	0.80	0.89
Rahal [18]	C15/20	3,47%	Laboratory	1-3-7-14-28 days	0.74	0.75
Rahal [18]	C20/25	3,47%	Laboratory	1-3-7-14-28 days	0.65	0.66
Rahal [18]	C25/30	3,47%	Laboratory	1-3-7-14-28 days	0.70	0.69
Rahal [18]	C30/37	3,47%	Laboratory	1-3-7-14-28 days	0.67	0.71
Rahal [18]	C40/50	3,47%	Laboratory	1-3-7-14-28 days	0.60	0.70
Hani et al. [64]	C15/20	3,34%	Laboratory	7-14-28 days	0.63	0.75
Hani et al. [64]	C25/30	3.34%	Laboratory	7-14-28 days	0.70	0.67
Hani et al. [64]	C40/50	3.34%	Laboratory	7-14-28 days	0.57	0.61
Kou et al. [17]	C35/45	6,20%	Industrial	1-4-7-28 days	0.73	0.75
Kou et al. [17]	C35/45	5,60%	Industrial	1-4-7-28 days	0.76	0.75
Kou et al. [17]	C35/45	3,77%	Industrial	1-4-7-28 days	0.64	0.75
Brandes and Kurama [38]	C40/50	8,16%	Industrial	1-3-7-28 days	0.85	0.85
Sim and Park [65]	C35/45	6,45%	Unknown	3-7-28 days	0.76	0.77
Gonzalez and Etxeberria [46]	C80/95	5,91%	Laboratory	1-7-28 days	0.88	0.89
Olorunsogo [67]	C25/30	Unknown	Industrial	3-7-28 days	0.53	0.46
Gao et al. [68]	C25/30	3.94%	Laboratory	7-28 days	0.85	0.80

4.1.1.2 Influence of curing conditions on strength development of RAC

The effect of the curing conditions on the strength development of RAC can be a significant factor in the process of using RCA in precast structural concrete. The curing conditions are known to modify the mechanical behaviour of concrete, especially where tension forces are applied [69]. The ability of RCA to absorb more water influences the water transport inside the concrete, as well as the water transport between the concrete and the surrounding environment [70]. From a practical point of view, the precast concrete elements are often not cured in moist conditions. They are primarily cured in ambient air in the precast plant. The temperature and humidity in a precast plant can vary according to variations in the weather conditions and seasons. Therefore, it is essential to know whether RAC reacts differently to non-standard curing conditions than NAC.

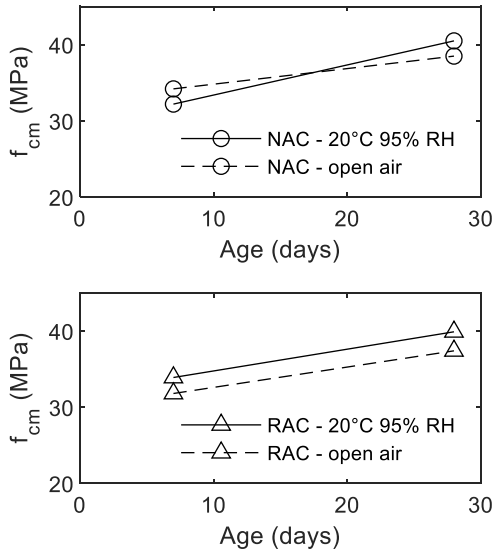


Figure 4-1 – Influence of curing condition on strength development (adapted from [71])

The curing conditions have been varied in several studies on concrete with RCA. Ravindrarajah et al. [72] reported the results of an experimental campaign investigating the mechanical properties of RAC, subjected to both water- and air-curing. It was observed that the presence of RCA results in a decreasing compressive strength, independent of the curing condition. Furthermore, the water-cured samples had higher strengths compared to specimens cured in the air. The difference in strength between specimens subjected to both curing conditions increases with the use of RCA in the mixtures. The strength development of concrete cured in air is not affected by the presence of RCA. On the contrary, RAC cured in water showed a slower strength development in the first seven days after concrete production. Fonseca et al. [70] reported the results of a series of tests where the influence of the curing conditions on the mechanical properties of RAC was examined. The compressive strength of RAC does not seem to be affected differently by the different curing conditions than that of NAC. The difference in the compressive strength of samples subjected to different curing conditions is relatively small. Similar results were also reported by Gayarre et al. [71]. In their research, two RCA types were used, and the replacement percentage of the RCA was also varied. The compressive strength at 28 days was higher with water-cured samples cured than air-cured samples, as shown in Figure 4-1. However, the results show that the strength development is only slightly affected by the curing method or RCA presence. Abdel-Hay [73] reported results of a similar experimental study to evaluate the effects of RCA on the compressive strength of concrete under different curing conditions. The aforementioned phenomenon of a higher strength of concrete cured in water accounts for RAC and NAC. The test results show that the strength development of RAC in the first 3 to 7 days is faster than concrete with natural aggregates for both air-cured and water-cured samples.

The aforementioned studies show that the curing conditions influence the strength development of RAC. However, no clear tendency can be drawn from the influence of RCA on the strength development of concrete under different curing conditions. The curing method is an essential aspect of the technology transfer from lab tests to actual applications. Lab research mainly uses standard curing conditions like water-curing, while precast elements are often cured in ambient air conditions. Therefore, specific studies are needed to understand

better the influence of RCA on the strength development of concrete under different curing conditions.

4.1.2 Test program

A test program is designed to assess the influence of RCA on the early-age mechanical behaviour of precast concrete under different curing conditions since this is directly important for the concrete plant efficiency.

4.1.2.1 Mix design

The precast concrete mixture with a design strength class of C50/60 used in this experimental campaign is described in Section 3.2.1. The coarse fraction of the aggregates is replaced by several replacement percentages of RCA, 0, 10, 20, 50 and 100%$_{V/V}$, respectively. The influence of RCA on the early-age mechanical properties of this mixture is tested by the five mixtures shown in Table 3-4. The concrete is produced in the Laboratory of Civil Engineering, KU Leuven Campus Bruges, according to the mixing procedure described in Section 3.3.1.

4.1.2.2 Test procedure

For each concrete mixture, 108 cubes (150x150x150 mm^3) and 36 cylinders (150x300 mm^3) were cast in six batches. 36 cubes were air-cured in a laboratory room with a temperature of 20±2 °C and relative humidity of 60-70 %. The other 72 cubes and 36 cylinders were cured in water in a climate room of 20±2 °C. To investigate early-age mechanical behaviour, different mixtures were tested at a curing age of 1, 2, 3, 7, 14 and 28. For each curing age, six cubes subjected to air-curing and six cubes subjected to water-curing were tested on the compressive strength [74] using a universal compressive strength machine with a maximum load of 5000kN. Furthermore, six water-cured cubes were tested on each curing age to determine the splitting tensile strength [75], and six water-cured cylinders were tested to determine the static elastic modulus [76].

4.1.3 Test results and discussions

4.1.3.1 Workability

The workability of the mixtures is tested by measuring the slump value according to NBN EN 12350-2 [57]; the mean value and standard deviation of the slump value of the different batches are shown in Table 4-2. The results show that an S4 slump class is reached, as the design considerations prescribe. Figure 4-2 shows the workability of RAC0 and RAC100.

Table 4-2 – Workability of the concrete mixtures

	RAC0	RAC10	RAC20	RAC50	RAC100
Mean slump value (mm)	190	185	190	195	190
Standard deviation (mm)	10	10	10	20	10



Figure 4-2 – Workability of RAC0 (left) and RAC100 (right)

4.1.3.2 Compressive strength

The mean compressive strength $f_{cm,cub}(t)$ measured on cubes subjected to water-curing are shown in Figure 4-3 and Table 4-3. The concrete mixtures with RCA

show a similar compressive strength development to the reference concrete. The compressive strength increase mainly occurs in the first seven days, irrespective of the replacement percentage of the RCA. In other words, the strength increase of the concrete mixtures from 7 to 28 days is minimal. After seven days, 92% of the 28-day strength is reached for RAC0 and RAC100, as shown in Table 4-4. The age factor at 2 and 3 days shows that the strength development of RAC100 is slightly slower than RAC0, as shown in Table 4-4. These results confirm previous results from [61] and [18].

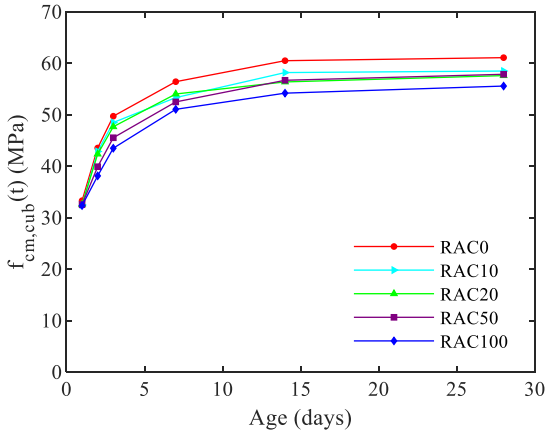


Figure 4-3 - Development of compressive strength with time, cured in water, mean values

Table 4-3 - Development of compressive strength with time, cured in water, mean values and standard deviation (MPa)

	$f_{cm,cub,1d}$ ($s_{fcm,cub,1d}$)	$f_{cm,cub,2d}$ ($s_{fcm,cub,2d}$)	$f_{cm,cub,3d}$ ($s_{fcm,cub,3d}$)	$f_{cm,cub,7d}$ ($s_{fcm,cub,7d}$)	$f_{cm,cub,14d}$ ($s_{fcm,cub,14d}$)	$f_{cm,cub}$ ($s_{fcm,cub}$)
RAC0	33.31	43.55	49.72	56.43	60.51	61.10
	(1.42)	(2.18)	(2.76)	(2.40)	(2.26)	(2.06)
RAC10	32.48	42.87	48.50	53.34	58.20	58.50
	(1.34)	(1.47)	(0.56)	(1.29)	(2.38)	(0.66)
RAC20	32.61	42.37	47.68	54.04	56.38	57.61
	(1.10)	(1.44)	(1.16)	(1.12)	(0.59)	(1.65)
RAC50	32.57	39.91	45.54	52.50	56.70	57.87
	(1.92)	(2.20)	(1.77)	(1.94)	(1.69)	(2.39)
RAC100	32.36	38.13	43.51	51.06	54.20	55.58
	(1.04)	(1.95)	(1.99)	(2.05)	(1.36)	(1.67)

Table 4-4 - Relative strength development (age factor) of RAC0 and RAC100 cured in air and water

	$f_{cm,cub,1d}/f_{cm,cub}$	$f_{cm,cub,2d}/f_{cm,cub}$	$f_{cm,cub,3d}/f_{cm,cub}$	$f_{cm,cub,7d}/f_{cm,cub}$	$f_{cm,cub,14d}/f_{cm,cub}$	$f_{cm,cub}/f_{cm,cub}$
RAC0 – water-cured	0.55	0.71	0.81	0.92	0.99	1
RAC100 – water-cured	0.56	0.69	0.78	0.92	0.98	1
RAC0 – air-cured	0.59	0.76	0.84	0.94	0.99	1
RAC100 – air-cured	0.59	0.71	0.79	0.92	0.99	1

Figure 4-3 also indicates that for each curing age, the compressive strength of the concrete mixtures decreases as the substitution of natural aggregates by RCA increases. The strength reduction of RAC is often considered to result from an increased porosity and a lower quality of the attached mortar in the RCA in the concrete [17,46]. This negative effect becomes less considerable with the improvement of the RCA quality. The minor reduction in the compressive strength at a curing age of 28 days of 4, 6, 5 and 9% of RAC10, RAC20, RAC50 and RAC100, respectively, compared to RAC0, is mainly due to the use of high-quality RCA. The target strength class C50/60 is not reached in these lab tests. However, the precast plant from which the industrial mixture originates, reports a higher strength with the same mixture in industrial conditions. More information is given in Chapter 7.

The mean test results of the compressive strength $f_{cm,cub}(t)$ measured on cubes that are air-cured are shown in Figure 4-4 and Table 4-5. RAC0 and RAC20 have a similar strength development and 28-day compressive strength, while RAC100 has a lower strength (-5%) than the reference concrete. From Table 4-4 can be seen that the age factor of the compressive strength at three days is 0.84 for RAC0 - air-cured, while this value is 0.79 for the mixture RAC100 - air-cured. Therefore the strength development at three days in air-curing for RAC100 is slower than that of RAC0, confirming the results of [18,61].

Table 4-5 - Development of compressive strength in time, air-cured, mean values and standard deviation (MPa)

	$f_{cm,cub,1d}$ ($s_{fcm,cub,1d}$)	$f_{cm,cub,2d}$ ($s_{fcm,cub,2d}$)	$f_{cm,cub,3d}$ ($s_{fcm,cub,3d}$)	$f_{cm,cub,7d}$ ($s_{fcm,cub,7d}$)	$f_{cm,cub,14d}$ ($s_{fcm,cub,14d}$)	$f_{cm,cub}$ ($s_{fcm,cub}$)
RAC0	34.46 (1.60)	44.51 (1.30)	48.84 (1.76)	54.76 (1.39)	58.08 (1.27)	58.46 (0.99)
RAC20	34.40 (0.88)	43.74 (1.42)	49.35 (0.70)	55.03 (0.95)	57.63 (1.56)	57.66 (2.16)
RAC100	32.59 (1.25)	39.42 (1.27)	43.96 (2.08)	51.27 (1.12)	55.03 (2.16)	55.56 (2.39)

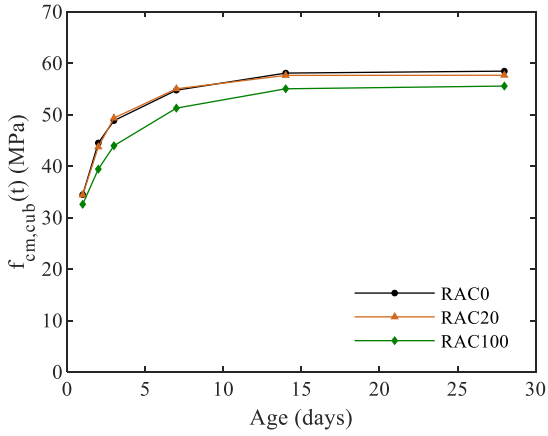


Figure 4-4 - Development of compressive strength in time, air-cured, mean values

Influence of curing conditions on strength development of RAC

Figure 4-5 compares the strength development of the compressive strength of the concrete under different curing conditions: water-curing (20°C) and air-curing (20°C – 60-70 % RH). The compressive strength is 5% lower for air-cured samples of RAC0 than that of the water-cured samples at 28 days, as shown in Figure 4-5-a. A general tendency of higher strength for water-cured samples has been concluded in literature [70–73]. The hydration of cement continues as long as free water is available. Air-cured samples of RAC0 generally have less water available because of evaporation, resulting in a lower strength than water-cured samples, as Figure 4-5 shows. The presence of absorption water inside the RCA in the concrete composition is responsible for an internal curing effect [62]. In

air-cured samples of RAC100, extra water released from the RCA to the fresh mortar paste keeps the hydration going. Therefore, a very similar compressive strength development is achieved for RAC100 under both curing conditions, as shown in Figure 4-5-b.

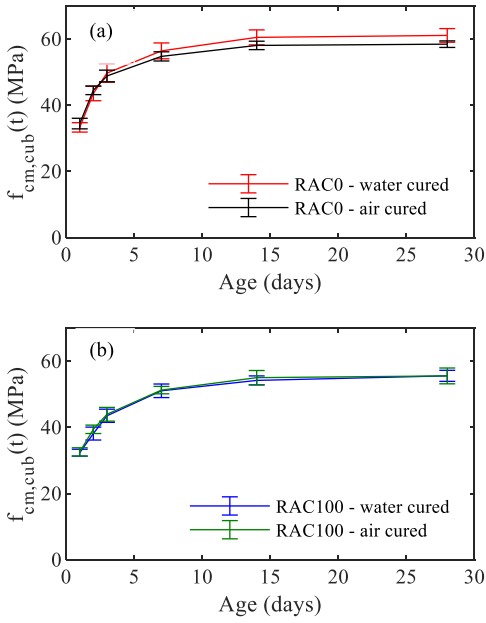


Figure 4-5 - Development of compressive strength of RAC0 (a) and RAC100 (b) in time, water-cured vs air-cured samples

Relative strength development of RAC

The relative strength development, mentioned in Table 4-4, also described as the age factor, is visually shown in Figure 4-6. The high-quality RCA slightly influence the age factor at two and three days. The mixtures with a higher replacement percentage have a lower age factor, indicating a slower strength development in the early ages. The influence of RCA on the age factor of the concrete at 7 and 14 days is negligible. Previous studies [17,18,61–64,66] also indicated a possible difference in strength development for up to 7 days.

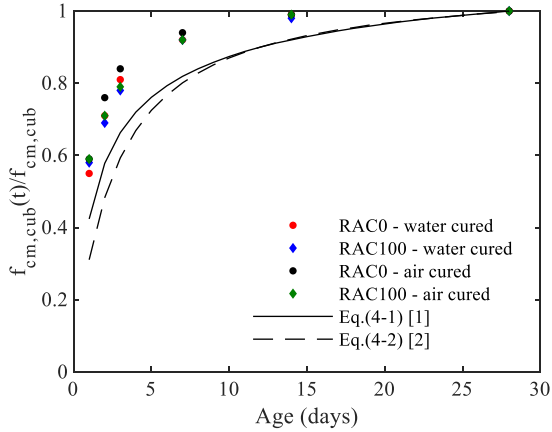


Figure 4-6 - Age factor for compressive strength of concrete vs time in water- and air-curing

The age factor at a certain age can be predicted in different ways, as stated in several standards [1,2]. In EN 1992-1-1 [1], the compressive strength of concrete at any time $f_{cm}(t)$ is calculated as

$$f_{cm}(t) = [\beta_{cc}(t)]f_{cm} \qquad (4\text{-}1)$$

in which

$\beta_{cc}(t) = \exp\left\{s\left[1-\left(\frac{28}{t}\right)^{\frac{1}{2}}\right]\right\}$; s = 0.20 for rapid hardening cement.

In ACI 209 [2], the following equation is recommended to predict the compressive strength of concrete at any time $f_{cm}(t)$:

$$f_{cm}(t) = \frac{t}{a+b\cdot t} f_{cm} \qquad (4\text{-}2)$$

where a and b are parameters depending on the used type of cement and the curing condition. For concrete with rapid hardening cement and moisture-cured concrete specimens, a = 2.30, b = 0.92.

A comparison of the predictions of Eq.(4-1) and Eq.(4-2) with the test data is given in Figure 4-6 under the assumption that Eq.(4-1) and Eq.(4-2) can be used to predict the strength development of concrete cubes. From the figure is clear that both models underestimate the development of the compressive

strength of the concretes with time. Even though Eq.(4-1) and Eq.(4-2) are not designed for other curing conditions than standard curing, it is clear that the strength development of the air-cured samples can also be predicted by Eq.(4-1) and Eq.(4-2).

4.1.3.3 Splitting tensile strength

The use of RCA in concrete influences the tensile strength differently compared with the effect on the compressive strength. The RCA generally have an angular shape and a porous surface, resulting in a stronger bond between RCA and the fresh mortar [77]. This has a positive effect on the tensile strength of concrete. Poon et al. [77] investigated the effect of the microstructure of ITZ on the strength properties of RAC. They reported that the ITZ between RCA and mortar is denser than the ITZ between natural aggregates and mortar due to RCA's porous surface. This results in a better bond between mortar and RCA. This effect is significant in evaluating the tensile strength of RAC [78].

The mean measured data on the splitting tensile strength $f_{ctm,sp}$ of the concrete mixtures at each curing age are presented in Figure 4-7 and Table 4-6. All the samples are water-cured. The tests are carried out according to EN 12390-6 [75]. From Figure 4-7, it is clear that the $f_{ctm,sp}$ of each concrete mixture increases with the curing time. At a curing age of 28 days, the splitting tensile strength of the mixture RAC10 and RAC20 slightly increases (+4% and +5%) in comparison to RAC0. The measured splitting tensile strengths of the other two mixtures, i.e., RAC50 and RAC100, are only 2% smaller than the reference concrete. The compressive strength results have shown that the effect of RCA is minimised due to the high quality of the RCA. The effect of RCA is negligible in terms of the tensile splitting strength. The negative effect of the porous attached mortar of the RCA on the mechanical properties is countered by a better bond between the mortar and the RCA [78].

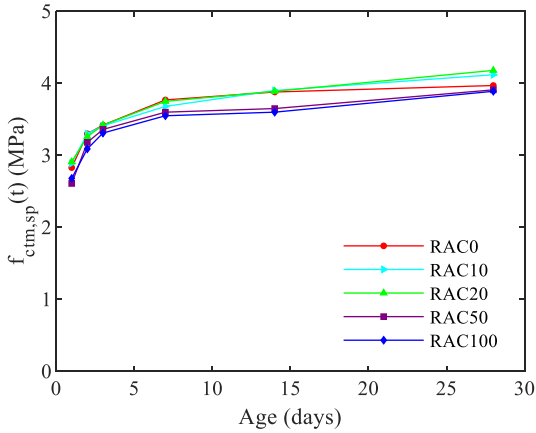


Figure 4-7 - Tensile splitting strength development of RAC, mean values

Table 4-6 – Tensile splitting strength development of RAC, mean value and standard deviation (MPa)

	$f_{ctm,sp,1d}$ ($s_{fctm,sp,1d}$)	$f_{ctm,sp,2d}$ ($s_{fctm,sp,2d}$)	$f_{ctm,sp,3d}$ ($s_{fctm,sp,3d}$)	$f_{ctm,sp,7d}$ ($s_{fctm,sp,7d}$)	$f_{ctm,sp,14d}$ ($s_{fctm,sp,14d}$)	$f_{ctm,sp}$ ($s_{fctm,sp}$)
RAC0	2.83 (0.22)	3.30 (0.14)	3.42 (0.16)	3.77 (0.30)	3.88 (0.27)	3.97 (0.23)
RAC10	2.89 (0.23)	3.30 (0.16)	3.41 (0.16)	3.68 (0.27)	3.90 (0.11)	4.12 (0.33)
RAC20	2.91 (0.19)	3.26 (0.28)	3.42 (0.24)	3.75 (0.17)	3.89 (0.11)	4.18 (0.16)
RAC50	2.61 (0.15)	3.18 (0.30)	3.36 (0.14)	3.60 (0.21)	3.65 (0.07)	3.91 (0.32)
RAC100	2.68 (0.15)	3.09 (0.27)	3.31 (0.21)	3.55 (0.24)	3.60 (0.08)	3.89 (0.31)

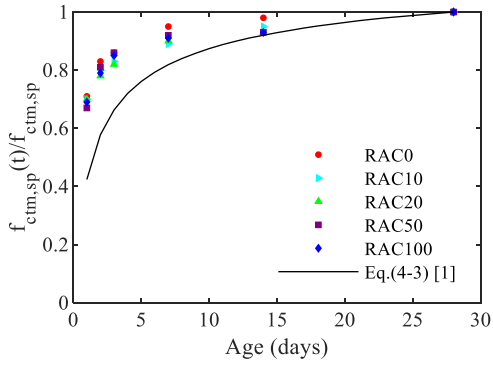


Figure 4-8 - Age factor for the tensile splitting strength development of RAC in time

Figure 4-8 presents the age factor for the splitting tensile strength of each concrete mixture. The data reveal no clear tendency on the influence of RCA on the splitting tensile strength development. The random differences between the mixtures are minor and can be neglected.

In EN 1992-1-1 [1], Eq.(4-3) is used to describe the relation between the splitting tensile strength of concrete at any time $f_{ctm,sp}(t)$ and that at a curing age of 28 days, $f_{ctm,sp}$.

$$f_{ctm,sp}(t) = [\beta_{cc}(t)]^{\alpha} f_{ctm,sp} \qquad (4\text{-}3)$$

in which, $\beta_{cc}(t)$ is defined as in Eq.(4-1); α is a parameter, for $t < 28d$, $\alpha = 1.0$. In Figure 4-8, the predictions of Eq.(4-3) are compared with the test data. Eq.(4-3) is conservative towards the test data. As the strength development is not affected by the presence of RCA in the mixtures, Eq.(4-3) can also be used as a predictive equation for the tensile strength development for both NAC and RAC.

The splitting tensile strength is important for analysing and designing concrete members and elements. It is generally predicted based on the compressive strength. Figure 4-9 reports the ratio of the $f_{ctm,sp}/f_{cm,cub}$ in function of the compressive strength of all the mixtures tested under different curing ages. The

ratio of the $f_{ctm,sp}/f_{cm,cub}$ is affected by the compressive strength $f_{cm,cub}$. The increase in splitting tensile strength occurs at a much lower rate than the increase in compressive strength. This is similar to the findings of other researchers [79,80].

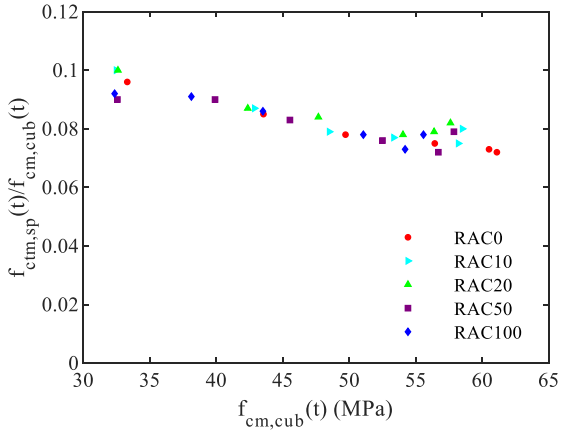


Figure 4-9 - Ratio of splitting tensile strength ($f_{ctm,sp}(t)$) to compressive strength ($f_{cm,cub}(t)$) versus compressive strength

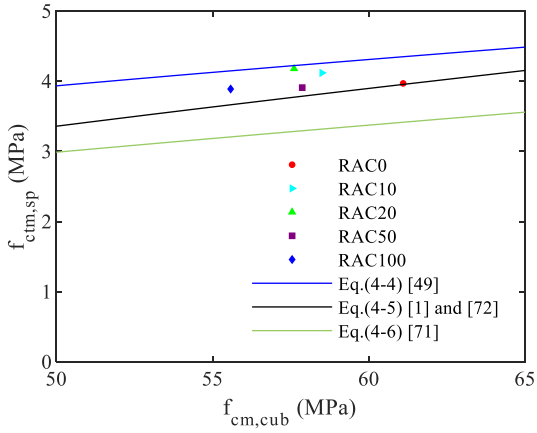


Figure 4-10 - Relation between splitting tensile strength ($f_{ctm,sp}$) and compressive strength ($f_{cm,cub}$) at 28 days

Figure 4-10 shows the relation between these two parameters for the test data on 28 days, as most of the prediction equations in the existing standards are focussed on the relation between f_{cm} and $f_{ctm,sp}$ on 28 days [78]. Each point represents the relation between the average value of six compressive tests and six tensile splitting tests on samples of the same mixture. The two types of strength are positively correlated without being considerably affected by the presence of RCA. The data from this research is compared with the prescribed relations in different standards. ACI 318-14 [40], EN 1992-1-1 [1], and JSCE [81] give their own predictive equations in Eq.(4-4), Eq.(4-5) and Eq.(4-6) respectively. *fib* Model Code [82] describes the same equation as EN 1992-1-1 [1]. Following conversions between different parameters were used to correctly apply the equations: $f_{ctm,sp} = f_{ctm,sp,cyl}/0.9$ [75]; $f_{ctm} = f_{ctm,sp}$ [82]; $f_{cm} = 0.8 \times f_{cm,cub}$ [1]. When the test data is compared with the predictions from different prescribed relations, suitable for this range of concrete strengths, the equation from JSCE [81] underestimates the splitting tensile strength when calculated based on the compressive strength. The other two equations accurately describe the relation between splitting tensile strength and compressive strength of the concrete tested in this study. The sum of squares, used to determine the best fit, confirms that statement. The sum of squares is the cumulative squared difference between the test data and the prediction. The sum of squares between the test data and the predictions from Eq.(4-4), Eq.(4-5) and Eq.(4-6) is 0.34; 0.32 and 2.58, respectively. Eq.(4-4) and Eq.(4-5) can be used to accurately predict this relation, Eq.(4-6) gives a conservative and safer relation.

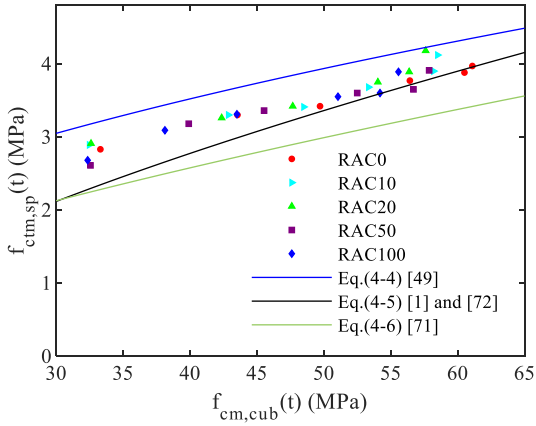


Figure 4-11 - Relation between $f_{cm,cub}(t)$ and $f_{ctm,sp}(t)$ for a curing age of 1, 2, 3, 7, 14 and 28 days

$$f_{ctm} = 0.56\ (f_{cm})^{0.5} \tag{4-4}$$

$$f_{ctm} = 0.3\ (f_{cm} - 8)^{2/3} \tag{4-5}$$

$$f_{ctm,sp} = 0.23\ (f_{cm})^{2/3} \tag{4-6}$$

The analysis of the relation between $f_{cm,cub}(t)$ and $f_{ctm,sp}(t)$ at early ages is more complicated due to the fast development of the mechanical properties due to the hydration process of cement. Figure 4-9 shows changes in this relation at early curing ages. Figure 4-11 shows the relation between $f_{cm,cub}(t)$ and $f_{ctm,sp}(t)$ at the age of 1, 2, 3, 7, 14 and 28 days, respectively. A larger scattering is noticed at a curing age of 1 day, corresponding to the data points with a compressive strength of less than 35 MPa. This is due to the fast-evolving properties of the mortar fraction in the concrete. In this regard, the experimental results from the samples with curing ages of 2 to 28 days are compared with the predictive equations Eq.(4-4), Eq.(4-5) and Eq.(4-6). Even though Eq.(4-5) and Eq.(4-6) are meant to predict the behaviour of concrete at 28 days, it might be important to know whether these equations can be applied to concrete at early

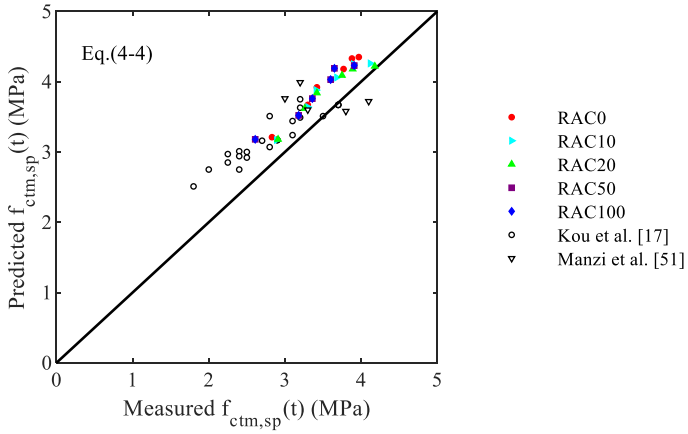
curing ages as precast elements are often subjected to high loads on an early age.

Figure 4-12 presents a comparison of the predictive equations Eq.(4-4), Eq.(4-5) and Eq.(4-6) with the measured data. This figure also includes previous test data on the relation between $f_{cm,cub}(t)$ and $f_{ctm,sp}(t)$ on different testing ages reported in [17,61]. The figure indicates that Eq.(4-4) overestimates the tensile splitting strength, while Eq.(4-5) and Eq.(4-6) slightly underestimate the test data for the tensile splitting strength.

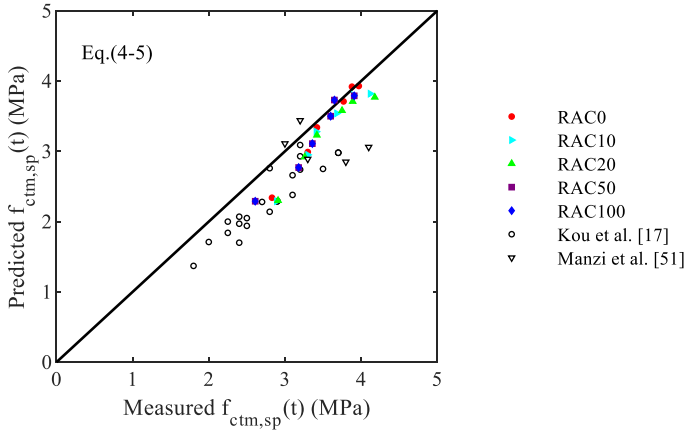
An accurate model to predict the relation between f_c and $f_{ct,sp}$ regardless of the curing age and RCA content is essential to precast concrete for the aforementioned reasons. From Figure 4-10 and Figure 4-11 can be concluded that the RCA content does not affect the relation between f_c and $f_{ct,sp}$ differently than NAC, confirming the statements of Silva et al. [78]. The scatter in the relation between the test data of $f_{cm,cub}(t)$ and $f_{ctm,sp}(t)$ is higher at one day curing age and therefore will not be considered in further calculations. Based on the regression analysis of the present data, the following equation is proposed to estimate the $f_{ctm,sp}(t)$ based on the compressive strength for a curing age of 2 days to 28 days.

$$f_{ctm,sp}(t) = 0.36\ (f_{cm}(t))^{0.59} \qquad (4\text{-}7)$$

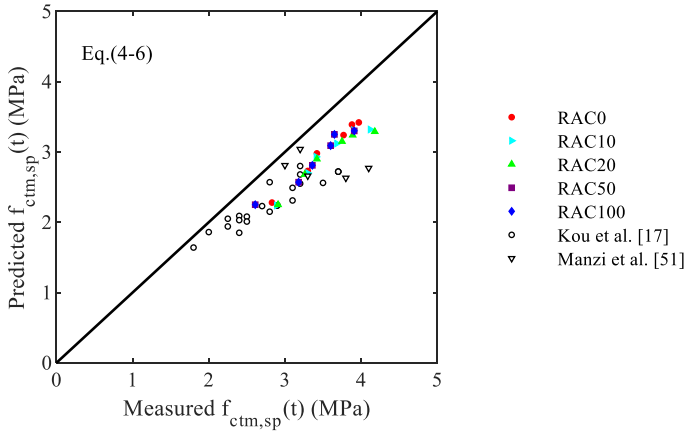
A comparison between the experimental data and the prediction of Eq.(4-7) is shown in Figure 4-13. The proposed equation gives a better description of all the test data (correlation coefficient R = 0.94) than those in Figure 4-12.



(a) Eq.(4-4) proposed by [40]



(b) Eq.(4-5) proposed by [1] and [82]



(c) Eq.(4-6) proposed by [81]

Figure 4-12 – Comparison between predictions of existing models for $f_{ctm,sp}(t)$ with test data: (a) Eq.(4-4) from ACI 318-14 [40], (b) Eq.(4-5) from EN 1992-1-1 [1], and (c) Eq.(4-6) from JSCE [81]

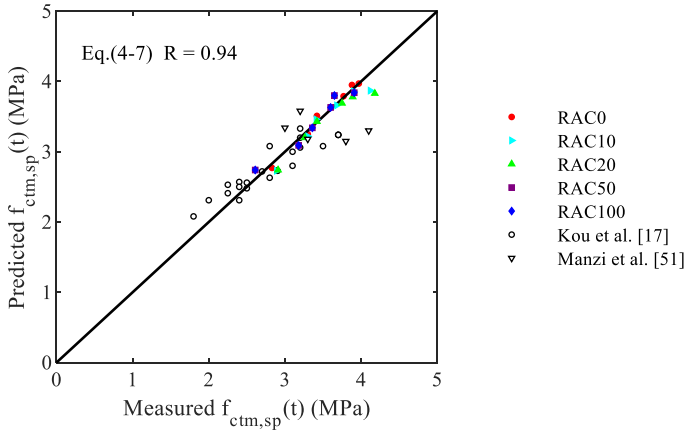


Figure 4-13 - Proposed relation between $f_{cm}(t)$ and $f_{ctm,sp}(t)$: predictions vs test data

4.1.3.4 Modulus of elasticity

It is commonly accepted that RCA in concrete reduces the modulus of elasticity, E_c, of the material [83]. An extensive literature review from Lye et al. [84]

concluded an average E-modulus reduction of 16% at 100% RCA replacement. However, this includes all types and strengths of concrete. The tested $E_{cm}(t)$ of the mixtures in this study is presented in Figure 4-14 and Table 4-7. The figure shows that the measured $E_{cm}(t)$ value of each concrete mixture increases in time. The increase mainly occurs in the first seven days. At any curing age, the $E_{cm}(t)$ of RAC is lower than that of NAC. A higher percentage of the RCA results in lower $E_{cm}(t)$ of the RAC. At the curing age of 28 days, the measured E_{cm} of the concrete mixture RAC10, RAC20, RAC50, and RAC100 is 96, 99, 96 and 91% of that of the reference concrete RAC0, respectively. This is consistent with the test results reported in previous studies [83,85].

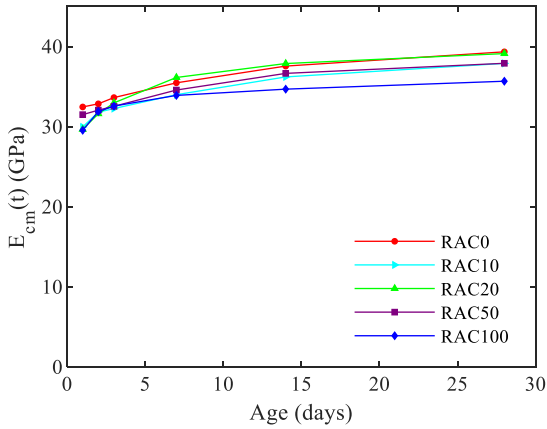


Figure 4-14 - Development of modulus of elasticity of concrete in time, mean values

Table 4-7 - Development of modulus of elasticity of concrete in time, mean values and standard deviation (GPa)

	$E_{cm,1d}$ ($s_{Ecm,1d}$)	$E_{cm,2d}$ ($s_{Ecm,2d}$)	$E_{cm,3d}$ ($s_{Ecm,3d}$)	$E_{cm,7d}$ ($s_{Ecm,7d}$)	$E_{cm,14d}$ ($s_{Ecm,14d}$)	E_{cm} (s_{Ecm})
RAC0	32.46 (1.59)	32.85 (1.74)	33.63 (1.45)	35.47 (0.98)	37.56 (0.65)	39.34 (0.92)
RAC10	29.97 (0.85)	31.81 (0.76)	32.26 (0.45)	33.99 (1.46)	36.21 (0.51)	37.90 (1.39)
RAC20	29.66 (1.22)	31.61 (0.27)	32.97 (0.90)	36.14 (0.79)	37.88 (1.49)	39.12 (0.82)
RAC50	31.50 (0.92)	32.06 (0.35)	32.54 (1.00)	34.57 (0.76)	36.65 (0.59)	37.91 (0.66)

RAC100	29.55 (1.93)	31.88 (0.91)	32.57 (1.39)	33.90 (1.78)	34.68 (0.58)	35.67 (0.56)

The age factor of the modulus of elasticity of each concrete mixture, defined as the ratio of the modulus of elasticity at any time t, i.e., $E_{cm}(t)$, to the modulus of elasticity at 28 days E_{cm}, is given in Figure 4-15. The age factor of the E-modulus is not much influenced by the incorporation of RCA in the concrete.

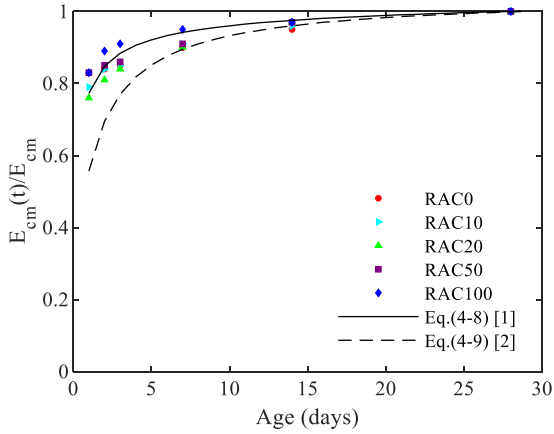


Figure 4-15 - Age factor for modulus of elasticity of concrete vs time

EN 1992-1-1 [1] recommended using Eq.(4-8) to estimate the $E_{cm}(t)$ of concrete based on E_{cm}.

$$E_{cm}(t) = [\beta_{cc}(t)]^{0.3} E_{cm} \qquad (4\text{-}8)$$

in which $\beta_{cc}(t)$ is the same function as defined in Eq.(4-1).

In ACI 209 [2], Eq.(4-9) is used to describe the relation between the $E_{cm}(t)$ and E_{cm} for concrete.

$$E_{cm}(t) = \sqrt{\frac{t}{a+b\cdot t}}\, E_{cm} \qquad (4\text{-}9)$$

where a and b are parameters that are defined in Eq.(4-2).

The predictions of Eq.(4-8) and Eq.(4-9) are also shown and compared to the experimental data in Figure 4-15. The model in Eq.(4-8) from EN 1992-1-1 [1] provides a relatively accurate description of the development of the $E_{cm}(t)$ of concrete with time, while Eq.(4-9) from ACI 209 [2] is more conservative towards the development of $E_{cm}(t)$ in time, especially for that before seven days.

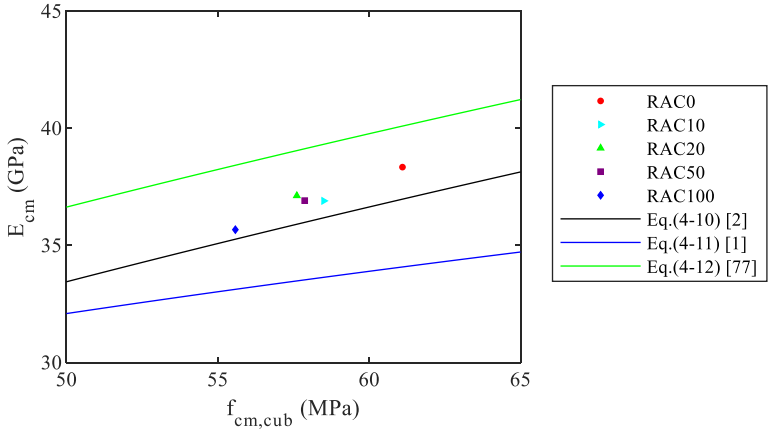


Figure 4-16 - Relation between E_{cm} and $f_{cm,cub}$ on 28 days

$$E_{cm}(t) = 4.73\ f_{cm}(t)^{0.5} \tag{4-10}$$

$$E_{cm} = 19.8\ (f_{cm}/10)^{0.3} \tag{4-11}$$

$$E_{cm} = 6.3\ f_{cm}^{0.45} \tag{4-12}$$

Different equations for estimating the E_{cm} of concrete (with and without RCA) have been developed in past studies [85,86]. Most equations are mainly based on the 28-day compressive strength. Figure 4-16 presents the relation of E_{cm} and $f_{cm,cub}$, applied on the test data on 28 days. The experimental data are compared to the predictions from standards ACI 209 – Eq.(4-10) [2], EN1992-1-1 – Eq.(4-11) [1] and JCI – Eq.(4-12) [87]. As illustrated in Figure 4-16, the equation in JCI overestimates the E_{cm}, the equations from EN1992-1-1

underestimates the data, and the equation from ACI 209 is relatively close to the data. In ACI 209 [2], Eq.(4-10) is proposed to estimate the E_{cm} of normal weight concrete at any time based on the compressive strength at the same age. This equation is compared with the dataset in this project in Figure 4-17. A good relation between the data and Eq.(4-10) can be concluded. Only the values of the specimen tested at one day, which have the lowest compressive strength, show a different relation between $E_{cm}(t)$ and $f_{cm,cub}(t)$. This is consistent to the results in Figure 4-11, where the relation between $f_{cm}(t)$ and $f_{ctm,sp}(t)$ of the tests at 1 day also shows a larger scattering.

fib Model Code [82] defines the relation between elastic modulus and compressive strength of concrete by Eq.(4-13):

$$E_{cm} = 21.5\ \alpha_E\ (f_{cm}/10)^{0.33} \qquad (4\text{-}13)$$

In Eq.(4-13), α_E is defined as a factor accounting for the influence of the different aggregate types on the relation between compressive strength and elastic modulus. The E_c of concrete can be increased by 20% or decreased by 30% by varying the type of aggregates [82]. Consequently, RCA will also affect this value. The values of α_E for the tested concrete mixtures are calculated using the average values of E_{cm} and f_{cm} of each mixture in Eq.(4-13). The calculated values of α_E are shown in Table 4-8. It is clear that the α_E-value decreases with a higher percentage of RCA in the mixture.

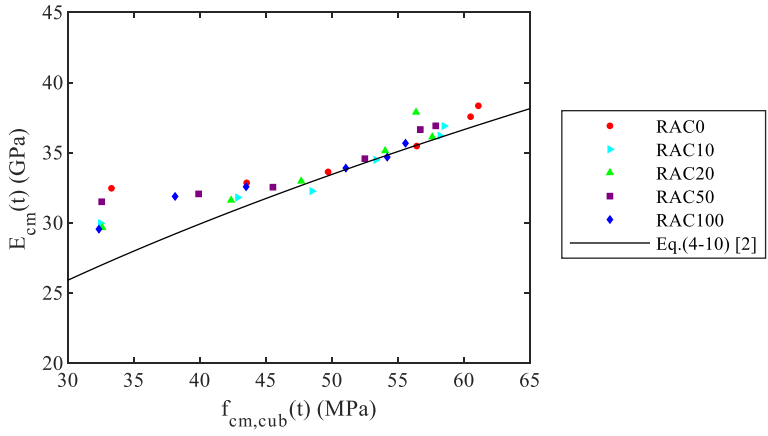


Figure 4-17 - Relation between $E_{cm}(t)$ and $f_{cm,cub}(t)$ for all curing ages

Table 4-8 - α_E for different mixtures

Mixtures	α_E
RAC0	1.00
RAC10	0.98
RAC20	1.01
RAC50	0.98
RAC100	0.94

4.2 Strength and modulus of elasticity of self-compacting precast structural concrete with high-quality RCA at early ages

The early-age mechanical properties of the self-compacting high-strength concrete with high-quality RCA are evaluated in this section. The development of strength properties and elastic modulus are compared with equations from different standards.

4.2.1 Test program

4.2.1.1 Mix design

These tests are designed to investigate the influence of high-quality RCA on the early-age mechanical properties of self-compacting concrete (SCC). An industrial SCC mixture with a design strength class of C50/60 is used as a reference, as described in Section 3.2.2. Different RCA replacement percentages, 0, 20 and 100%$_{V/V,}$ are used to replace the coarse fraction of the limestone in the original mixture. The three mixtures are shown in Table 3-5. The concrete is made in the Laboratory of Civil Engineering, KU Leuven Campus Bruges, according to the mixing procedure described in Section 3.3.2.

4.2.1.2 Test procedure

In total, 30 cubes (150x150x150 mm^3) and 30 cylinders (150x300 mm^3) for each concrete mixture were cast in three batches. The cubes and cylinders were cured in water in a climate room with a temperature of 20±2°C. The different mixtures were tested at a curing age of 1, 2, 3, 7 and 28 days, respectively, to investigate the early-age mechanical behaviour. Six water-cured cube samples were tested for each age to determine the compressive strength [74] using a universal compressive machine with a maximum load of 5000kN. Furthermore, six water-cured cylinders were tested on the compressive strength and static elastic modulus [76] for each age.

4.2.2 Test results

4.2.2.1 Workability

The workability of the mixtures was tested by measuring the slump flow value according to EN 12350-8 [88] and shown in Table 4-9. The measured slump flow values of the concrete mixtures are nearly equal for all mixtures due to the absorption water and superplasticizer of the type MasterEase 5035. This project focuses on the influence of high-quality RCA on the mechanical properties; hence no further tests on the workability and consistency of the self-compacting

concrete are done. Figure 4-18 shows the workability of RASCC0 and RASCC100 (fLTR).

Table 4-9 – Workability of RASCC

	RASCC0	RASCC20	RASCC100
Mean slump flow (mm)	760	750	760
Standard deviation (mm)	10	10	10



Figure 4-18 - Workability of RASCC0 and RASCC100

4.2.2.2 Compressive strength

The compressive strength development of the cubes and cylinders cured in water are shown in Figure 4-19, Table 4-10, Figure 4-20 and Table 4-11. The mean compressive strength of the three mixtures is close to each other, especially at curing ages of 7 and 28 days. However, some discrepancies can be noticed in the first three days after the concrete pouring. The compressive strength of RASCC100 is slightly lower than that of the other mixtures at a curing age of 28 days. A decreased compressive strength of 4% is observed when RCA replaces 100% of the coarse limestone fraction. This decrease is smaller compared to the observed decrease of 9% when RCA fully replaces the limestone in the

conventional concrete mixture, as shown in Figure 4-3. As explained in Section 3.2.1 and 3.2.2, the ratio of the coarse limestone fraction to the complete aggregate skeleton is smaller for SCC than for the conventional concrete. The coarse fraction of limestone is 45%$_{V/V}$ of the total aggregate skeleton for the conventional concrete, while this is only 27%$_{V/V}$ for SCC. This explains the slight decrease when RCA are fully incorporated into the SCC. This effect is also visible in Figure 4-21, where the age factor of the compressive strength of the different mixtures is shown. The strength development of RASCC0 and RASCC100 is nearly similar. A slight difference between RASCC0 and RASCC100 can be noticed in the first three days, similar to the results of the conventional concrete shown in Figure 4-6. For cubes and cylinders, the strength development evolves slower when RCA are incorporated in the mixtures. At the age of 7 and 28 days, the difference in strength development between RASCC0 and RASCC100 becomes negligible.

Table 4-10 - Compressive strength development of RASCC tested on cubes, mean values and standard deviation (MPa)

	$f_{cm,cub,1d}$ ($s_{fcm,cub,1d}$)	$f_{cm,cub,2d}$ ($s_{fcm,cub,2d}$)	$f_{cm,cub,3d}$ ($s_{fcm,cub,3d}$)	$f_{cm,cub,7d}$ ($s_{fcm,cub,7d}$)	$f_{cm,cub}$ ($s_{fcm,cub}$)
RASCC0	40.28 (1.03)	53.11 (1.21)	57.86 (2.15)	66.63 (2.49)	71.18 (2.32)
RASCC20	38.70 (1.21)	52.11 (1.95)	56.55 (0.58)	64.75 (2.00)	70.20 (2.73)
RASCC100	36.99 (1.58)	49.85 (2.65)	54.08 (1.92)	63.97 (0.82)	68.52 (2.46)

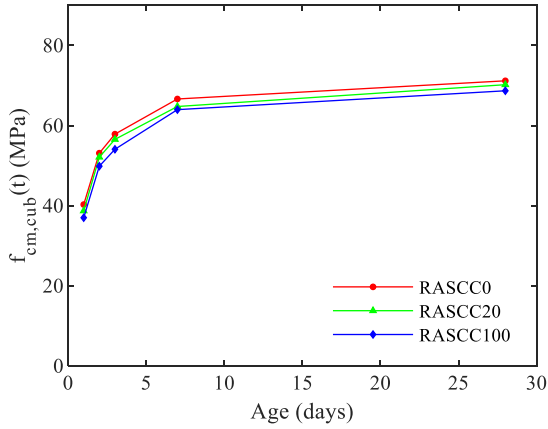


Figure 4-19 - Compressive strength development of RASCC tested on cubes, mean values

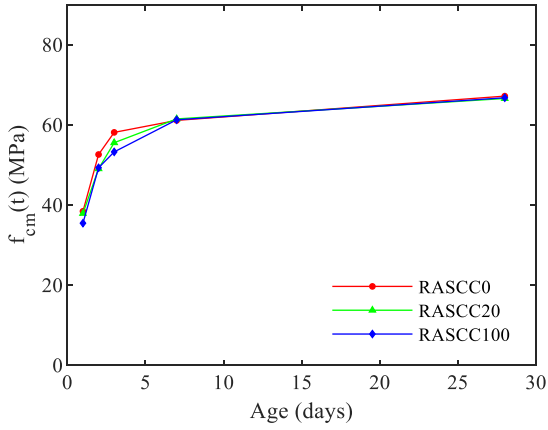


Figure 4-20 - Compressive strength development of RASCC tested on cylinders, mean values

Table 4-11 - Compressive strength development of RASCC tested on cylinders, mean values and standard deviation (MPa)

	$f_{cm,1d}$ ($s_{fcm,1d}$)	$f_{cm,2d}$ ($s_{fcm,2d}$)	$f_{cm,3d}$ ($s_{fcm,3d}$)	$f_{cm,7d}$ ($s_{fcm,7d}$)	f_{cm} (s_{fcm})
RASCC0	38.48 (0.96)	52.66 (1.48)	58.18 (1.04)	61.17 (2.44)	67.23 (2.16)
RASCC20	37.95 (1.14)	49.10 (2.58)	55.60 (2.01)	61.54 (2.12)	66.64 (2.07)
RASCC100	35.51 (2.01)	49.34 (1.88)	53.31 (2.21)	61.32 (2.33)	66.82 (3.39)

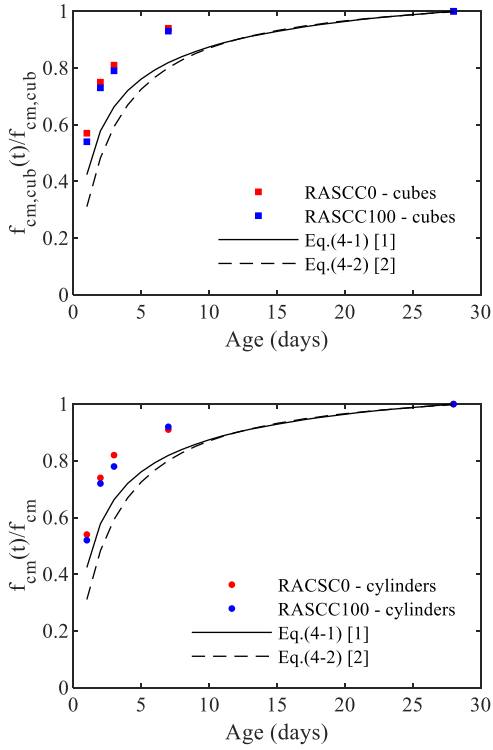


Figure 4-21 - Age factor for compressive strengths of cubes and cylinders in time

A comparison of the predictions of Eq.(4-1) and Eq.(4-2) with the test data for both cubes and cylinders are shown in Figure 4-21. Both models underestimate the development of the compressive strength of the concrete with time.

4.2.2.3 Static elastic modulus

The tests on the modulus of elasticity of concrete in this section were performed according to B15-203 [76]. The tested $E_{cm}(t)$ of the concrete are presented in Figure 4-22. Six cylinders are tested at each curing age, which is 1, 2, 3, 7 and 28 days. The measured $E_{cm}(t)$ values of each concrete mixture increase with the curing time. From the results can be concluded that the RCA has nearly no considerable effect on the static elastic modulus on 7 and 28 days. In the first three days, it seems that the static elastic modulus of RASCC100 is slightly lower than that of the other mixtures, although the difference is small. This is similar to the compressive strength results, where the presence of RCA results in slower development of the strength at a curing age of 2 and 3 days. This can also be noticed in Figure 4-23, where the age factor of the elastic modulus is shown. The elastic modulus development is slightly lower for RASCC100 than RASCC0 and RASCC20.

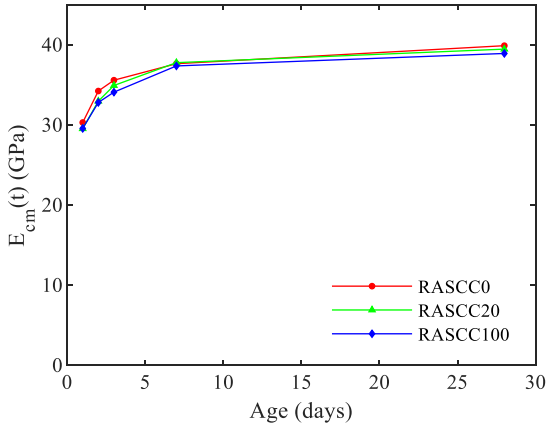


Figure 4-22 - Static elastic modulus development, mean values

Table 4-12 – Static elastic modulus development, mean values and standard deviation (GPa)

	$E_{cm,1d}$ ($s_{Ecm,1d}$)	$E_{cm,2d}$ ($s_{Ecm,2d}$)	$E_{cm,3d}$ ($s_{Ecm,3d}$)	$E_{cm,7d}$ ($s_{Ecm,7d}$)	E_{cm} (s_{Ecm})
RASCC0	30.29 (1.24)	34.22 (1.54)	35.57 (0.73)	37.63 (1.09)	39.88 (1.03)
RASCC20	29.52 (0.75)	32.95 (1.23)	34.91 (0.39)	37.76 (0.75)	39.45 (0.67)
RASCC100	29.55 (1.23)	32.79 (0.65)	34.08 (1.01)	37.35 (1.41)	38.91 (1.38)

The predictions of Eq.(4-8) and Eq.(4-9) are also shown and compared to the experimental data in Figure 4-23. The model in EN 1992-1-1 [1] provides a relatively accurate description of the development of the $E_{cm}(t)$ of concrete with time, while the equation in ACI 209 [2] is generally more conservative towards the increase of the $E_{cm}(t)$ with time, mainly before seven days.

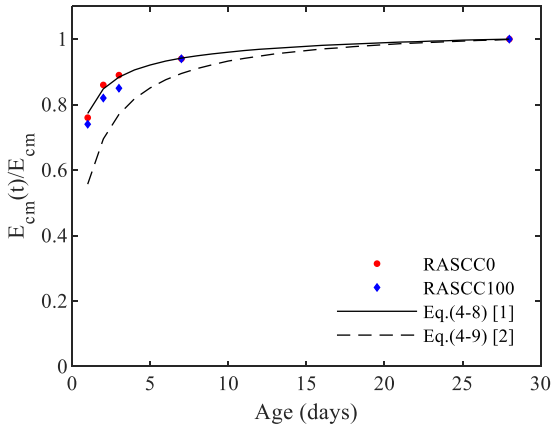


Figure 4-23 - Age factor elastic modulus

Figure 4-24 presents the relation of E_c and f_c, both tested on the same cylinder on 28 days. The test data is compared to Eq.(4-10) [2], Eq.(4-11) [1] and Eq.(4-12) [87]. The test results of E_c are found to be quite accurate determined by the equation in ACI 209 [2], while Eq.(4-12) from JCI [87] overestimates, and EN1992-1-1 [1] underestimates the relation between E_c and f_c.

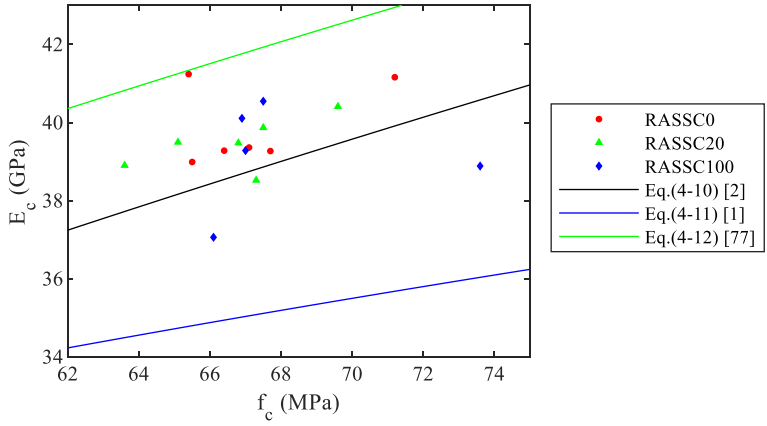


Figure 4-24 - Relation between elastic modulus and compressive strength at 28 days

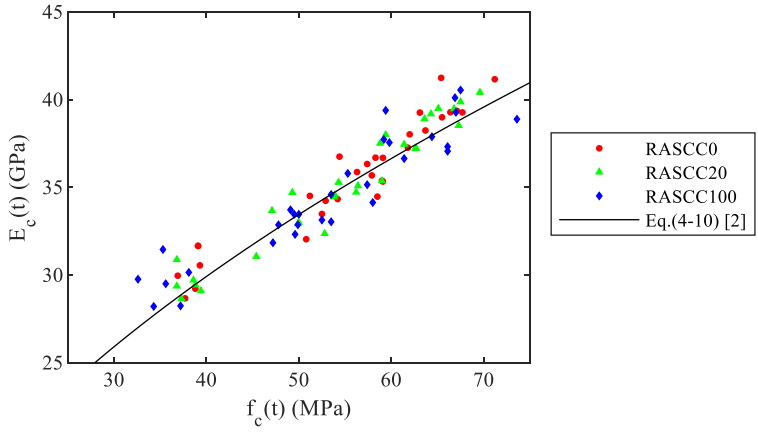


Figure 4-25 - Relation between elastic modulus and compressive strength

In ACI 209 [2], Eq.(4-10) is proposed to estimate the E_c of concrete at any time based on the concrete compressive strength at the same age for normal-weight concrete. This equation is compared with the dataset in this research in Figure 4-25. A good relation between the data and Eq.(4-10) can be concluded for all the curing ages, similar to the conventional concrete data. The good relation

between the predicted values based on Eq.(4-10) [2] and the average tested values is shown in Figure 4-26.

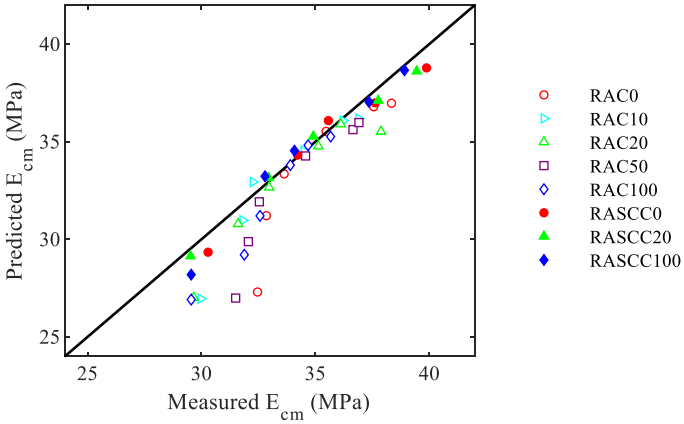


Figure 4-26 – Relation between predicted values and tested values

4.3 Conclusions

This chapter presents a systematic analysis of the laboratory test results on the mechanical properties of two concrete mixtures with high-quality RCA at early ages to understand the effect of RCA on the concrete strength development. A precast concrete mixture with workability class S4 and a precast self-compacting concrete mixture, both with a design strength class of C50/60, were tested at 1, 2, 3, 7, 14 and 28 days. The effect of the percentage of RCA, curing time and curing method on the compressive strength, tensile splitting strength, and modulus of elasticity of the concrete was investigated. Within the scope of this research, the following conclusions can be drawn.

The precast structural concrete mixture has decreased 9% in 28-day compressive strength when the coarse fraction of limestone is entirely replaced by high-quality RCA. The decrease in tensile splitting strength is negligible, while the elastic modulus has a similar tendency as for the compressive strength with a 9% decrease at 28 days. The high-quality RCA have only marginal effects on the mechanical properties of the self-compacting precast concrete mixture. This is due to the high quality of the RCA and the smaller coarse fraction replaced

by the RCA. The replaced fraction represents $27\%_{V/V}$ of the total aggregate skeleton, while the replaced fraction in the precast structural concrete described in Section 3.2.1 is $45\%_{V/V}$ of the total aggregate skeleton.

Incorporating RCA in the concrete mixtures negatively influences the compressive strength development in the first three days after the concrete production. The age factor of the compressive strength at the age of two and three days is for RAC100 2 – 5% lower than that for RAC0. This effect can be minimised by limiting the amount of RCA to $20\%_{V/V}$. The high-quality RCA do not influence the strength development at 7 to 28 days. No clear effect of RCA on the development of the tensile splitting strength and elastic modulus is observed. The curing condition has little effect on the concrete strength development where the coarse limestone fraction is entirely replaced by the high-quality RCA. This contradicts the NAC, where a higher strength is observed for water-cured samples than air-cured samples.

In practice, the compressive strength is the only strength parameter tested in precast concrete production. Therefore, it is necessary to have accurate models to calculate the tensile strength and elastic modulus based on the compressive strength of precast concrete. An adapted empirical model is proposed to estimate the splitting tensile strength of precast concrete based on the regression analysis of the present laboratory test data. The proposed equation gives a better description of all the test data (correlation coefficient $R = 0.94$) compared to other available models, regardless of the replacement percentage of RCA in the concrete. The model in ACI 209 [2] to predict the elastic modulus based on the compressive strength provides a satisfactory description of the test data in the present study. This model is designed to estimate the E_{cm} of normal weight concrete at any time based on the compressive strength at the same age. Based on the available test data, the incorporation of RCA does not influence the relation in this model.

Chapter V

Variability of the compressive strength of RAC

The precast industry is an expanding construction sector, following a strict quality control system to guarantee high-quality finished products. The use of RCA in precast concrete to enhance the sustainability and cost-effectiveness of the products has attracted the attention of the precast industry worldwide. However, for that purpose, both the qualities of the materials and the structural reliability of the precast RAC have to be guaranteed.

Even for RCA produced from a single source of concrete rubble, the variability in the composition and properties can be more significant than that of natural aggregates. This can increase the variability of the concrete properties, such as the compressive strength, slump, unit weight, air content, and so on when RCA are used, and finally, decrease the reliability of structural components and structures made of RAC.

This chapter presents a laboratory study on the variability of the compressive strength of concrete incorporating a high-quality RCA produced from a recycling plant in Flanders. The scattering in the RCA properties and its effect on the variability in conventional precast concrete and precast SCC's compressive strength are discussed.

5.1 Statistical analysis of the compressive strength of RAC

5.1.1 State of the art

The standard deviation and variance are the most used statistic values to evaluate the variability of a population or dataset. It measures the spread of different data points relative to their mean value. A higher spread of the data results in a higher standard deviation.

$$\text{Standard deviation } s = \sqrt{\frac{\sum_{i=1}^{n} (x_i - \overline{x})^2}{n - 1}} \qquad (5\text{-}1)$$

The standard deviation of an entire population 'σ' is a fixed parameter, while the standard deviation of a dataset 's' is a statistic parameter, calculated by Eq.(5-1). The standard deviation of an entire population is calculated in the same way as in Eq.(5-1), only replacing $n - 1$ in the denominator by n. In this way, the population standard deviation will be lower than the sample standard deviation.

The sample coefficient of variation (CoV) is defined as the ratio of the standard deviation to the mean value. In some cases, the CoV value is preferred to the standard deviation because it gives a relative value to the mean value. When comparing the variability between data sets with different concrete classes, the coefficient of variation can be a valuable alternative to the standard deviation since it is less affected by the strength level [89]. However, the coefficient of variation should not be used for data if the mean value is small; in this case, the coefficient of variation can have considerable variation with minor changes in the mean value.

The use of RCA in concrete can increase the variability in the concrete properties and thus reduce the structural reliability of RAC. The evaluation of concrete variability can be divided into **within-batch variability** and **batch-to-batch variability**. The former evaluates the scattering in concrete

properties within the same batch, while the latter evaluates the variability of concrete from different batches. In previous studies, the influence of using RCA on the variation in the compressive strength of concrete was mainly investigated. The results from earlier attempts (before 2000) have been extensively reviewed by Hansen [90], ACI Committee 555 [91] and Li [92]. From those results, the following findings can be observed:

- Under controlled conditions in the laboratory, the use of RCA does not significantly affect the CoV of the concrete compressive strength, provided that the same RCA is used throughout the series of tests [93–97]. This conclusion is mainly based on the evaluation of the within-batch variability.
- The use of RCA produced from concrete waste of different qualities can result in an increased CoV for the compressive strength of concrete than that of concrete with the same RCA [98,99]. In the test of De Pauw [98], RCA produced from 12 types of concrete waste of different qualities were employed, and the CoV for the concrete compressive strength was found to be 12%. Nishibayashi and Yamura [99] used seven RCA types produced by crushing old concretes from different sources. The test results indicated that the CoV for the compressive strength of the RAC increases compared with that of the concrete with natural aggregates. The former could be up to 20%, while the latter was below 10%, implying that the compressive strength of concrete with RCA varies more widely. It should be noted that the results were based on batch-to-batch variability tests.
- The use of industrially produced RCA can further increase the compressive strength variation of concrete, particularly when the RCA are from different recycling plants. This was confirmed by Hendriks [100], who reported that the CoV-value of the compressive strength is around 25% for RAC produced by various Dutch producers. Again this was based on batch-to-batch variability tests.

Those results provided helpful information about the potential risk related to the use of RCA in structural concrete, causing significant concerns in the implementation of RAC in engineering practice [101,102]. It should be pointed out that the majority of earlier (laboratory) tests were based on a relatively

small amount of test specimens; for instance, in [99], only 3 - 5 specimens were examined. This is not consistent with the needs of ACI Committee 214 [103], which required a sample size of more than 30 tests. More recently, it is permitted to estimate the within-batch variability based on at least ten same-age strength test results [103].

To better understand the effect of RCA on the variability of the compressive strength of concrete, several test programs have been carried out since 2000 following the recommendations of ACI Committee 214 [103]. Mukherjee et al. [104] examined the variability in the compressive strength of concrete with 100% RCA and natural aggregates under identical conditions based on a total of 20 cube specimens for each type of concrete. It was observed that the standard deviation of the compressive strength of the RAC is comparable to that of the reference concrete with natural aggregates; while the CoV for the compressive strength is higher for the RAC than that for the reference concrete since the mean compressive strength of the former is significantly lower than that of the latter. Rahal [18] carried out a similar study but with different strength grades, and it was also noticed that the variation in the compressive strength of RAC tends to increase compared to that of conventional concrete prepared with natural aggregates. Similar results were also presented by Henry et al. [105], who also found that a mix of RCA from multiple sources further increases the concrete compressive strength variability. On the contrary, the test results reported by Breccolotti and Materazzi [106] and Pacheco et al. [107] did not show a clear trend on the RCA's influence on the variability in the concrete compressive strength.

Many tests have shown that an entire substitution of natural aggregates with RCA can affect many concrete properties [90,91]. It is often recommended to limit the amount of RCA in concrete to minimise its possible influence [34,42]. Therefore, it is interesting to evaluate the influence of RCA replacement percentage on the variability of concrete compressive strength. Li [92] and Xiao et al. [108] performed an extensive laboratory study on this issue based on many test specimens. In their test programme, a total of 4 RCA replacement percentages, i.e., of 0, 30, 50 and 100%, were used, and for each concrete mixture, 75 - 100 specimens were tested. The individual test data for each

specimen can be found in Li [92]. The test results indicate that the use of the RCA does not have a considerable effect on the CoV for the 28d concrete compressive strength. The highest CoV appeared for concrete with intermediate RCA replacement percentage of 30 and 50%. It was also found that the normal and lognormal distribution models can be applied to fit the concrete compressive strength at a 95% confidence level, regardless of the RCA replacement percentage. Later, Henry et al. [105], Breccolotti and Materazzi [106] as well as Pacheco et al. [107] also reported similar results. In contrast, Etxeberria et al. [109] reported that the CoV for the compressive strength of concrete increases with the increase of the RCA replacement percentage. Moreover, Breccolotti and Materazzi [110] observed that the CoV for concrete compressive strength could either increase or decrease when the replacement percentage of RCA increases, depending on the quality of the used RCA.

Other factors influencing the variability in the compressive strength of RAC were also investigated in the literature. Henry et al. [105] tested the influence of the W/C ratio on the variation of the RAC compressive strength. It was found that lower W/C ratios yield a slightly higher variation in the compressive strength. Jayasuriya et al. [111] investigated the effect of the attached mortar content on the variability in the mechanical properties of RAC using two-dimensional finite element analysis. The simulation results revealed that the concrete compressive strength variability increases as the adhered mortar content increases. Ulloa et al. [112] presented a study on the effect of the composition and properties of RCA on the variability of concrete compressive strength. The test results confirmed that contaminants in the RCA tend to increase the compressive strength variability. The authors stated that an increase of the RCA replacement percentage does not significantly affect the within-batch variability of the concrete compressive strength while it is responsible for the batch-to-batch variability of concrete compressive strength even when the same RCA is used. However, it should be noted that those results were based on relatively limited test specimens (3 cylinders).

From existing test data in the literature, it seems challenging to reach a consistent conclusion about the effect of RCA on the variability of the compressive strength of concrete. One reason for those inconsistent results is the

difference in the RCA quality. Another reason, which may be more important, is the direct comparison of the CoV or standard deviation for the concrete compressive strength without critically considering the difference of its mean values for the concrete with and without RCA. It is widely accepted that the CoV for the concrete compressive strength is dependent on its mean value; namely, the CoV for the concrete compressive strength reduces as the mean compressive strength value increases. Due to this reason, it is better to compare the CoV for the compressive strength of RAC with that of conventional concrete with identical compressive strength to examine better the effect of RCA on the variability of concrete compressive strength. This is, unfortunately, not the case in the majority of past studies.

On the other hand, despite previous efforts, one crucial question remains unanswered: what is the relation between RCA variability and the variability of the compressive strength of concrete? In particular, most studies focused on the 28-d compressive strength of the concrete, while no research has been done on the concrete strength at early ages (before three days). Laboratory tests, for example in [18,113], have shown that the use of RCA has a considerable impact on the development of the concrete compressive strength at early ages, which is possibly caused by the absorption water inside the RCA influencing the interfacial transition zone (ITZ) between RCA and mortar in concrete. As a result, the influence of RCA on the variability of the concrete compressive strength at early ages can be different from that of the 28-days compressive strength. The information about the variability of 1-day compressive strength of concrete incorporating RCA would help evaluate the structural reliability of this material's elements and structures at early ages. With this aim, the present chapter provides an extensive laboratory study on the influence of RCA on the variability of the 1-day and 28-days compressive strength of concrete using two types of concrete mixtures with different RCA replacement percentages. Particular focus is paid on the variability of the employed RCA's properties and its link to the concrete compressive strength variability at 1 and 28 days, respectively.

5.1.2 Test program

5.1.2.1 Test design

This study aims to gain insight into the effect of RCA on the 1-day and 28-days concrete compressive strength variability. As discussed above, test results indicated that the CoV-value of the concrete compressive strength is not independent of the mean strength value; consequently, it is more realistic to compare the CoV for the compressive strength of RCA concrete with that of conventional concrete of similar compressive strength. Many studies have shown that RCA used in concrete often decreases its compressive strength [90]. This means, if an identical mix proportion is used for both concrete with and without RCA, different mean compressive strength values would appear. Dhir et al. [114] and Xiao et al. [108] recommend adjusting the W/C ratio in the mixture to achieve a similar compressive strength for RAC and conventional NAC. With the RCA content increase, the W/C ratio needs to be reduced. Provided that the water in the concrete mixture is kept constant, a higher cement amount is required when RCA are incorporated. Previous test results by the authors [113] indicated that using a high-quality RCA does not cause considerable reductions in the concrete compressive strength with the increase of the RCA. Therefore, no adjustments in the W/C ratio are necessary to prevent considerable changes in the concrete compressive strength.

The conventional precast concrete mixture and the SCC precast mixture, as described in Section 3.2, are used in this experimental campaign. The concrete is made in the Laboratory of Civil Engineering, KU Leuven Campus Bruges, according to the mixing procedure described in Section 3.3.1. The compressive strength was tested according to NBN EN 12390-3 [74] using a universal compressive strength machine with a maximal load of 5000kN.

- The coarse fraction of the aggregates in the **conventional precast concrete mixture** is replaced by several replacement percentages of RCA, which are 0, 10, 20, 50 and 100%$_{V/V}$, respectively. The five mixtures are shown in Table 3-4. For each concrete mixture, 36 specimens from three batches and 40 cubes (150x150x150 mm^3) from two batches are made to examine the influence of RCA on the

variability of the 1-day and 28-days concrete compressive strength, respectively. The number of tests for each concrete mixture meets the requirements of ACI Committee 214 [103]. These mixtures are furtherly described as RAC mixtures.

- Different RCA replacement percentages, which are 0, 20 and 100%$_{V/V}$, are used to replace the coarse limestone fraction in the original **SCC precast mixture**. The three mixtures are shown in Table 3-5. Twenty cubes for each SCC mixture were cast in one batch. The cubes were cured in water in a climate room with a temperature of 20±2 °C. The different mixtures were tested at a standard curing age of 28 days. These mixtures are furtherly described as RASCC mixtures.

5.1.2.2 Materials and their properties

The materials used in the mix proportions in Table 3-4 are described in Section 3.1. As mentioned above, this section aims to examine the effect of incorporating the RCA on the concrete compressive strength variability. For this purpose, the variation in the properties of each aggregate is firstly tested, including the oven-dry density (ρ_{rd}), 24h water absorption (WA_{24}) and particle size distribution. Seven or eight samples are taken from different areas in a pile of high-quality RCA and limestone. The aggregates' ρ_{rd} and WA_{24}, tested according to NBN EN 1097-6 [52], are shown in Table 5-1, in which the number of samples (n) is also shown.

Table 5-1 - Oven-dry density and water absorption of the aggregates

Item	Aggregate	n	Min.	Max.	Mean	s	CoV (%)
ρ_{rd} (kg/m^3)	Sand 0-2 mm	8	2559	2595	2580	12	0.45
	Limestone 2-6 mm	7	2669	2697	2683	10	0.37
	Limestone 6-14 mm	8	2627	2662	2649	11	0.42
	RCA 6-14mm	7	2352	2409	2381	18	0.75

WA_{24} (%)	Sand 0-2 mm	8	0.22	0.55	0.40	0.10	23.97
	Limestone 2-6 mm	7	0.25	0.56	0.45	0.11	25.05
	Limestone 6-14 mm	8	0.58	0.85	0.78	0.09	11.25
	RCA 6-14 mm	8	4.07	4.90	4.46	0.26	5.86

Table 5-1 shows that the RCA's measured oven-dry density is about 10% lower than that of the natural aggregates, generally due to the influence of the attached mortar. However, the standard deviation of the RCA's oven-dry density is considerably higher than that of the natural aggregates, indicating a non-uniform distribution of the attached mortar on the RCA. These results are in line with the test data of Pickel [115], who also concluded a higher standard deviation of the oven-dry density of the RCA compared to natural aggregates. The RCA's water absorption is higher than that of the natural aggregates, which is a fact that has been widely accepted and is resulted from the effect of the porous attached mortar on the RCA [43]. However, it is interesting to notice that the CoV of the RCA's water absorption is much lower than that of the natural aggregates, which is caused by the significantly lower mean value for the water absorption of the natural aggregates. Due to this reason, a direct comparison of the CoV of the measured WA_{24} of the RAC and that of the natural aggregates is less meaningful. A better evaluation of the RCA variability is the standard deviation, which is considerably higher for RCA than natural aggregates.

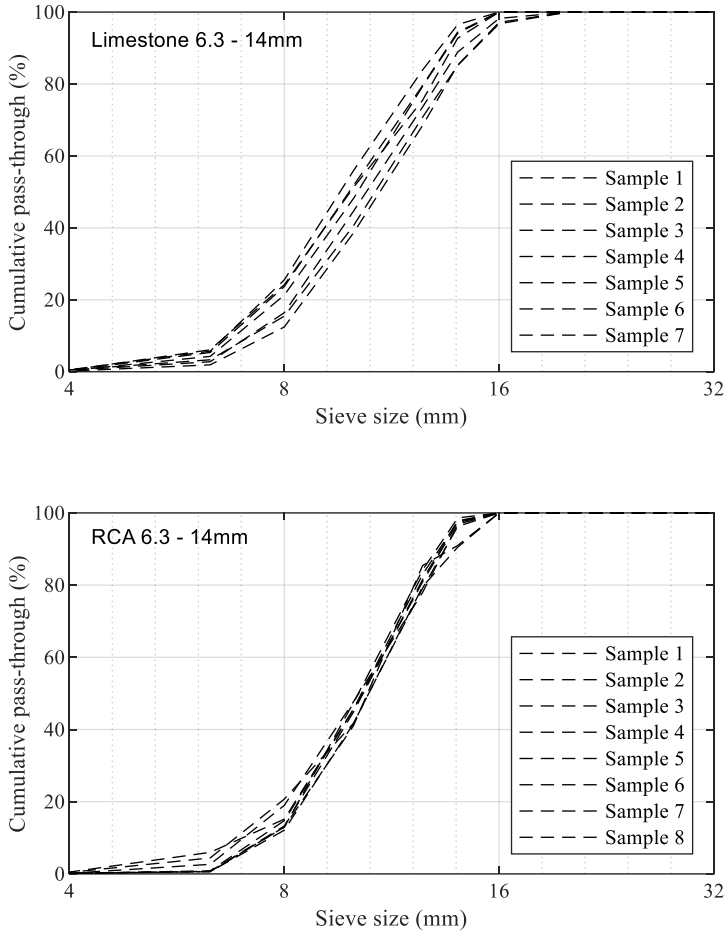


Figure 5-1 – Variation in size distribution between limestone aggregates 6.3 - 14 mm (top) and RCA 6.3 - 14 mm (bottom)

Figure 5-1 presents the measured size distribution of limestone 6.3-14 mm and RCA 6-14 mm according to EN 933-1 [51]. The grading curve of limestone 6.3-14 mm is similar to the RCA 6-14 mm, as already shown in Figure 3-2. Figure 5-1 shows that the scattering on the size distribution of limestone 6.3-14 mm is higher than that of the RCA. The difference between the minimal and maximal

value of the cumulative pass-through on several sieve sizes for limestone 6.3-14 mm and RCA 6.3-14 mm, shown in Table 5-2, confirms this statement. The fact that the limestone aggregates are produced on a large scale and have endured transportation can explain this. The fraction of the high-quality RCA are sieved in the lab.

Table 5-2 – Minimal and maximal value of cumulative pass-through of limestone 6.3-14 mm and RCA 6.3-14 mm

	Limestone 6.3 - 14 mm		RCA 6.3 – 14mm	
Sieve size (mm)	Max value (%)	Min value (%)	Max value (%)	Min value (%)
6.3 mm	1.91	6.10	0.62	5.97
8 mm	12.43	25.52	12.09	20.70
10 mm	38.56	55.88	40.99	47.64
12.5 mm	68.48	83.89	77.97	85.41
14 mm	85.18	96.52	90.51	98.61

5.1.3 Test results

5.1.3.1 Variability of the properties of the aggregates after the mixing process

The properties of each type of aggregate are elaborately tested to link the variability in aggregate properties to the variability of the concrete compressive strength. However, the aggregate properties can be affected during the mixing process because of the mechanical grinding of the aggregates. The evolution of the aggregate properties during the mixing process is tested in this regard.



Figure 5-2 – Samples taken from the mixer

The conventional precast concrete mixtures from Table 3-4 are made according to the mixing procedure described in Section 3.3.1. Eight samples from each mixture were taken directly from the mixer after the complete mixing process, as shown in Figure 5-2. The cement was washed out on a sieve of 90µm. The coarse fraction 6.3 - 14mm was sieved and tested on ρ_{RD} and WA_{24}. The mean values and standard deviations are shown in Figure 5-3 as the **orange bars**. To assess the evolution of the properties of the aggregate mixture during the mixing process, a reference aggregate mixture was mixed by hand, excluding any effects of aggregate grinding and other effects from the interaction between mixer and aggregates. The aggregate skeleton is compiled according to the mix proportions and mixed by hand for 5 minutes. Eight samples per mixture are taken and tested on ρ_{RD} and WA_{24}. The mean values and standard deviation are shown in Figure 5-3 as the **blue bars**.

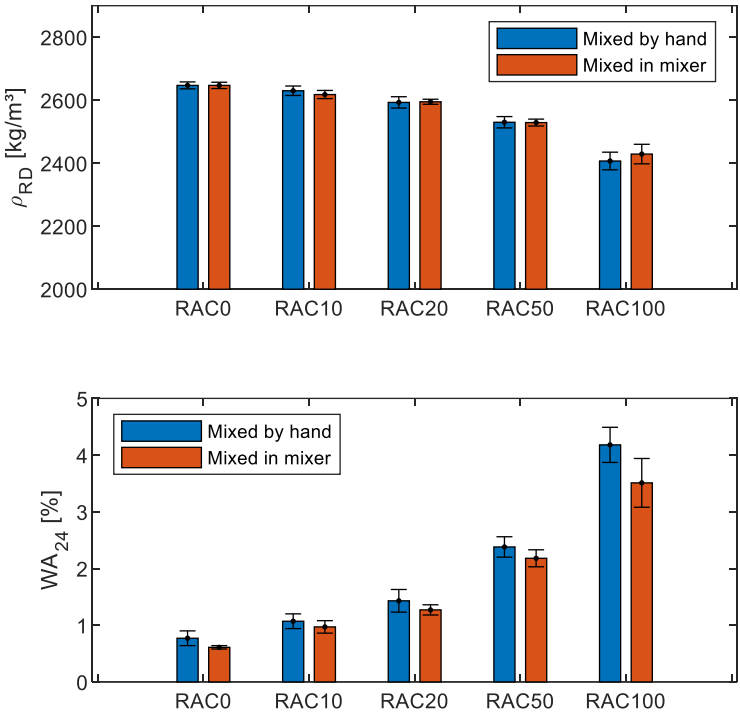


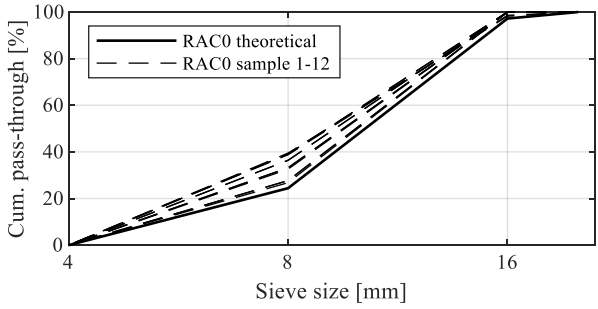
Figure 5-3 – Change of aggregate properties during the mixing process

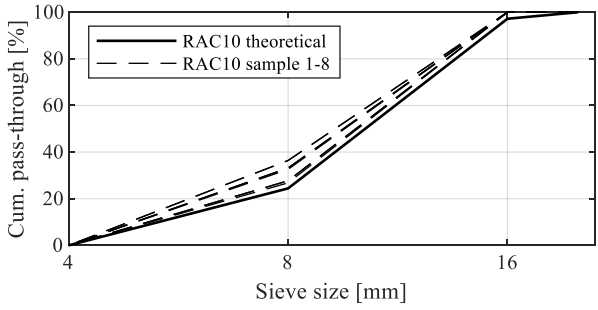
The oven-dry density of the coarse aggregate skeleton decreases as the RCA replacement percentage increases because the RCA have a smaller oven-dry density compared to limestone. The mixing process does not change the values of ρ_{RD} remarkably for RCA replacement percentages up to 50%$_{V/V}$. The oven-dry density of the coarse aggregate skeleton of RAC100 slightly increases by 1% after being mixed in the laboratory mixer. The WA_{24} increases as more RCA are incorporated into the mixtures. It is a general observation that the WA_{24} of the coarse aggregate skeleton decreases when the aggregates are mixed in the laboratory mixer, especially for RAC100 (-19%). With the increased ϱRD, this phenomenon is caused by the mechanical friction between the RCA and the mixer, and the aggregates themselves. This friction causes the weaker attached

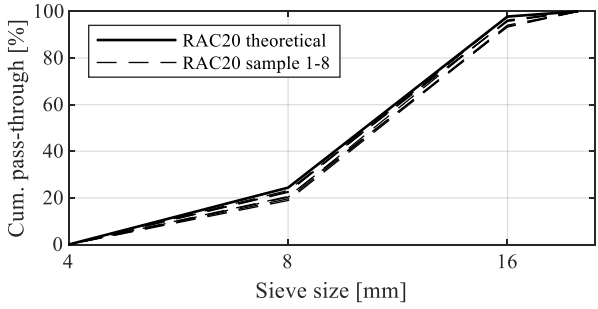
mortar to pulverise. Since the RCA are not pre-saturated, it is also possible that cement paste partially fills the pores of the RCA during the mixing process. This enclosed cement is not washed away, decreasing the WA_{24} and increasing the ρ_{RD} of the coarse aggregate skeleton.

The CoV-values of the measured properties are not considered since no comparisons can be made due to a significant difference in mean values. The standard deviation is used to assess the properties' variability during the mixing process. The standard deviation for ρ_{RD} is higher for RAC100 than the other mixtures. A similar tendency can be noticed for the WA_{24}. RAC0 only has natural aggregates in the mixture, which all have a similar water absorption and oven-dry density. Therefore, minor differences in the mix proportions of RAC0 will not result in significant differences in WA_{24} between the samples. RCA have a much higher WA_{24} than the natural aggregates, so minor variations in mix proportions can result in a higher standard deviation of the WA_{24} and ρ_{RD} of RAC100. The aforementioned non-uniform distribution of the attached mortar on the RCA also contributes to the higher standard deviation in Figure 5-3.

The size distribution of the coarse fraction of the aggregate skeleton is reported in Figure 5-4. The theoretical grading curve for each aggregate skeleton is composed using the measured values of the grading curves of the different values, as shown in Table 3-3, and the theoretical mix proportions, as shown in Table 3-4. This theoretical curve is compared to the measured grading curves of the coarse fraction of each aggregate skeleton to observe any differences in grain size or mix proportions of the aggregates. Except for RAC20, the coarse fraction of every mixture seems to have more particles smaller than 8 mm after the mixing process. This indicates the wearing of the aggregates due to friction during the mixing process. Any dust produced by the aggregates' friction is washed away with the cement. The measured coarse fraction of the aggregate skeleton of RAC20 is coarser than the theoretical value. Since the theoretical value is based on measured grading curves of different samples, it is possible that the used aggregates in the concrete mixture are coarser than the theoretical value should predict.


RAC0 theoretical
RAC0 sample 1-12
Cum. pass-through [%]
Sieve size [mm]
0
20
40
60
80
100
4
8
16


RAC10 theoretical
RAC10 sample 1-8
Cum. pass-through [%]
Sieve size [mm]
0
20
40
60
80
100
4
8
16


RAC20 theoretical
RAC20 sample 1-8
Cum. pass-through [%]
Sieve size [mm]
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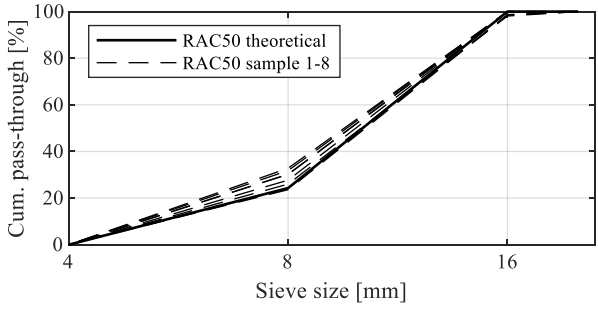
20
40
60
80
100
4
8
16


RAC50 theoretical
RAC50 sample 1-8
Cum. pass-through [%]
Sieve size [mm]
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20
40
60
80
100
4
8
16

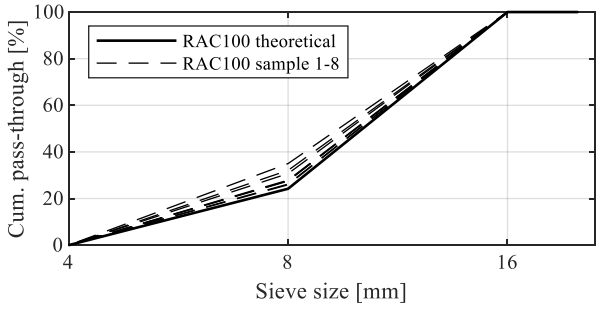


Figure 5-4 – Sieving curve after the mixing process

5.1.3.2 Workability

The workability of the RAC mixtures is tested by measuring the slump value according to NBN EN 12350-2 [57]; the mean value of three tests per mixture is shown in Table 5-3. The results show that an S4 slump class is reached.

Table 5-3 – Workability of RAC

	RAC0	RAC10	RAC20	RAC50	RAC100
Mean slump value (mm)	190	185	190	195	190
Standard deviation (mm)	10	10	10	20	10

The workability of the RASCC mixtures is tested by measuring the slump flow value, according to NBN EN 12350-8 [88]. The mean value of two tests per mixture is shown in Table 5-4. It is found in the tests that the measured slump flow values of the concrete mixtures are nearly equal for all mixtures. This is due to the addition of the absorption water and superplasticizer of the type MasterEase 5035. No further tests on the workability and consistency of the self-compacting concrete are done.

Table 5-4 – Workability of RASCC

	RASCC0	RASCC20	RASCC100
Mean slump flow (mm)	760	750	760
Standard deviation (mm)	10	10	10

5.1.3.3 Variability of the compressive strength of RAC

The evaluation of the variability in the compressive strength of concrete is divided into batch-to-batch variability and within-batch variability. The **overall variability** should be evaluated by considering both types. The **batch-to-batch variability** evaluates the variability in concrete properties between several batches of the same mixture. Causes of batch-to-batch variability can be accounted to minor differences in properties of ingredients, small changes in W/C ratio, variation in mixing and placing... [40] The **within-batch variability** evaluates the variability in properties of concrete within the same batch. These variations can be caused by a slight variation in mix proportions within one mixture, a difference in curing conditions or sample testing [40].

Table 5-5 shows the median compressive strength $\widetilde{f_{c,cub}}$, the mean compressive strength $f_{cm,cub}$, the standard deviation s and the coefficient of variation CoV of the compressive strength of the tests at one day, while Table 5-6 shows the same parameters of the tests at 28 days. For each mixture, the first line mentions the statistical parameters calculated by all the cubes of one mixture, while the following lines show the statistical parameters of each batch separately.

Batch-to-batch variability of the compressive strength

The batch-to-batch variability of the compressive strength is essential concerning RCA's use in precast concrete since a constant quality of the concrete among several batches is crucial in precast concrete production. Small changes in RCA properties used in different batches can lead to concrete

strength changes. Three batches of 12 cubes are made for each mixture tested at one day. The statistical parameters of the compressive strength of these three batches per mixture are shown in Table 5-5. The batch-to-batch variability is evaluated by comparing the mean values of the different batches of the same mixture. Figure 5-5 shows the mean compressive strength of the three batches per mixture. The spread on the mean values of different batches within one mixture seems slightly higher for RAC0 and RAC20, although the difference between maximum and minimum is still smaller than 2 MPa and within ranges of a typical standard deviation of the concrete's compressive strength. Thus, there is no clear tendency that the batch-to-batch variability on the compressive strength at one day is influenced by the use of the RCA in the mixtures.

The statistical parameters of the compressive strength tested on 28 days are shown in Table 5-6. Each mixture is tested in two batches of 20 cubes. The mean compressive strength values of the different batches are shown in Figure 5-6. There is also no clear tendency that the batch-to-batch variability on the compressive strength on 28 days is influenced using RCA in the mixtures. This can be explained by the fact that the RCA used in the different batches all originate from the same concrete waste. The properties of RCA from different origins might differ more, which can enlarge the batch-to-batch variability.

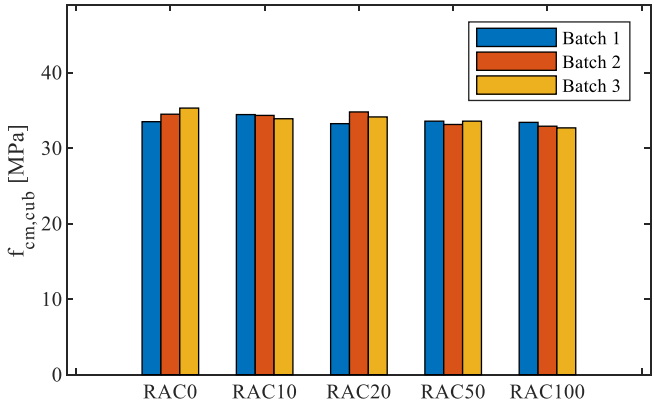


Figure 5-5 – Batch-to-batch variability of f_c of RAC at one day

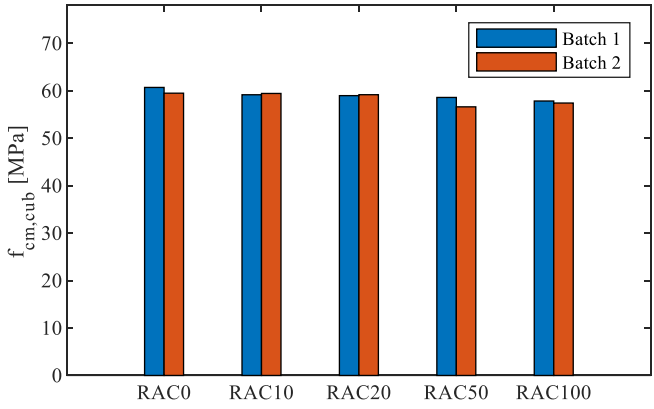


Figure 5-6 – Batch-to-batch variability of f_c of RAC at 28 days

Within-batch variability of the compressive strength

The within-batch variability evaluates the variability in compressive strength of concrete within the same batch. The results of the within-batch variability of the compressive tests at one day and 28 days are shown in Table 5-5 and Table 5-6, respectively. Although the standard deviation and CoV value of different batches of the same mixture varies, a clear tendency of an increasing within-batch variation on the test results with an increasing RCA content can be noticed for both curing ages. This is in accordance with the test results of [19] and [20].

Section 5.1.3.1 already shows that the standard deviation of ρ_{RD} and WA_{24} of the aggregate skeleton increases as RCA's replacement percentage increases. This indicates minor discrepancies in the mix proportions or uneven distributions of attached mortar on the RCA, which will lead to an under- or overestimation of the water absorption in different parts in the mixer. This leads to a local change in the W/C factor, increasing the within-batch variability of RAC.

Table 5-5 – Statistical parameters for $f_{c,cub}$ tested at one day

	$f_{c,max}$ (MPa)	$f_{c,min}$ (MPa)	$\widetilde{f_{c,cub}}$ (MPa)	$f_{cm,cub}$ (MPa)	s (MPa)	CoV (%)	$f_{cm,cub}/f_{cm,cub,RAC0}$
All batches RAC0	37,14	32,13	34,45	34,43	1,46	4,24	1
Batch 1: 12 cubes	*35.19*	*32.54*	*33.35*	*33.50*	*0.76*	*2.27*	
Batch 2: 12 cubes	*36.48*	*32.13*	*35.08*	*34.49*	*1.49*	*4.33*	
Batch 3: 12 cubes	*37.14*	*32.35*	*35.12*	*35.30*	*1.49*	*4.21*	
				Mean COV		*3.60*	
All batches RAC10	37,51	31,67	34,18	34,22	1,45	4,24	0,99
Batch 1: 12 cubes	*37.51*	*32.02*	*34.87*	*34.44*	*1.75*	*5.09*	
Batch 2: 12 cubes	*35.67*	*32.51*	*34.38*	*34.33*	*1.08*	*3.15*	
Batch 3: 12 cubes	*36.44*	*31.67*	*33.64*	*33.89*	*1.50*	*4.43*	
				Mean COV		*4.22*	
All batches RAC20	38,25	31,59	33.71	34,05	1,50	4,40	0,99
Batch 1: 12 cubes	*34.33*	*31.59*	*33.57*	*33.24*	*0.82*	*2.46*	
Batch 2: 12 cubes	*38.25*	*32.34*	*34.15*	*34.79*	*1.96*	*5.63*	
Batch 3: 12 cubes	*36.40*	*32.16*	*33.98*	*34.13*	*1.15*	*3.38*	
				Mean COV		*3.8 2*	
All batches RAC50	37,12	27.60	33,46	33,40	2,01	6,01	0,97
Batch 1: 12 cubes	*36.03*	*30.02*	*33.66*	*33.57*	*1.60*	*4.77*	
Batch 2: 12 cubes	*36.51*	*27.60*	*33.45*	*33.13*	*2.21*	*6.66*	
Batch 3: 12 cubes	*37.12*	*29.72*	*33.13*	*33.57*	*2.29*	*6.83*	
				Mean COV		*6.09*	
All batches RAC100	38,03	28,61	32,83	33,00	2,10	6,38	0,96
Batch 1: 12 cubes	*38.03*	*30.05*	*33.77*	*33.41*	*2.52*	*7.55*	
Batch 2: 12 cubes	*35.06*	*31.34*	*32.74*	*32.90*	*1.04*	*3.17*	
Batch 3: 12 cubes	*38.00*	*28.61*	*32.33*	*32.68*	*2.52*	*7.70*	
				Mean COV		*6.1 4*	

Table 5-6 - Statistical parameters for $f_{c,cub}$ tested at 28 days

	$f_{c,max}$ (MPa)	$f_{c,min}$ (MPa)	$\widetilde{f_{c,cub}}$ (MPa)	$f_{cm,cub}$ (MPa)	s (MPa)	CoV (%)	$f_{cm,cub}/f_{cm,cub,RAC0}$
All batches RAC0	64.46	57.16	59.55	60.16	1.92	3.17	1
Batch 1: 20 cubes	*63.22*	*58.62*	*60.54*	*60.71*	*1.58*	*2.60*	
Batch 2: 20 cubes	*64.46*	*57.16*	*58.81*	*59.54*	*2.06*	*3.46*	
				Mean COV		*3.03*	
All batches RAC10	64.13	52.80	59.29	59.32	2.06	3.47	0,99
Batch 1: 20 cubes	*63.16*	*52.80*	*59.39*	*59.17*	*2.01*	*3.39*	
Batch 2: 20 cubes	*64.13*	*54.90*	*59.21*	*59.44*	*2.11*	*3.55*	
				Mean COV		*3.47*	
All batches RAC20	64,42	55,71	59,12	59,13	1,80	3,05	0,98
Batch 1: 20 cubes	*60.82*	*56.45*	*59.46*	*58.98*	*1.34*	*2.27*	
Batch 2: 20 cubes	*64.42*	*55.71*	*58.65*	*59.18*	*2.17*	*3.67*	
				Mean COV		*2.97*	
All batches RAC50	62,72	51,09	58,07	57,76	2,83	4,92	0,97
Batch 1: 20 cubes	*62.72*	*53.13*	*58.56*	*58.60*	*2.32*	*3.97*	
Batch 2: 20 cubes	*61.96*	*51.09*	*56.44*	*56.62*	*3.22*	*5.68*	
				Mean COV		*4.83*	
All batches RAC100	62,54	52,21	57,61	57,61	2,55	4,43	0,96
Batch 1: 20 cubes	*62.54*	*52.21*	*58.19*	*57.85*	*2.93*	*5.06*	
Batch 2: 20 cubes	*60.05*	*52.44*	*58.00*	*57.42*	*2.10*	*3.66*	
				Mean COV		*4.36*	

Overall variability of compressive strength of RAC

Figure 5-7 shows the overall mean value and standard deviation for all the cubes' compressive strength of one mixture combined. The results show a similar tendency for the compressive strength on both testing ages. The compressive strength decreases 4% when RCA fully replace the natural coarse aggregates. According to the literature review from Silva et al. [39], this decrease is relatively small. The negative influence of RCA on the compressive strength is primarily due to the addition of a second interfacial transition zone (ITZ) in the concrete and the porosity of the attached mortar [77]. These negative influences are

minimised when the RCA quality is high by carefully selecting concrete waste and an optimised crushing process. In this case, the use of high-quality RCA only causes a minor decrease in the concrete compressive strength.

The overall variability of the compressive strength of RAC is a combination of the within-batch variability and the batch-to-batch variability. All test data for every mixture is combined, and the standard deviation and CoV values are used to assess the overall variability. It seems that the standard deviation stays similar for replacement percentages up to 20%$_{V/V}$. An increased CoV is observed for the compressive strength of concrete mixtures with 50 and 100%$_{V/V}$, both for a testing age of one and 28 days. It can be observed that the CoV-value of compressive tests on one day is generally higher than those of the tests on 28 days. This can be attributed to the fast-evolving mechanical properties of concrete at early ages, which is also discussed in Chapter 4. The influence of RCA on the mean values and variability of the compressive strength of concrete is relatively small. A statistical analysis is necessary to distinguish if the observed differences between the mixtures are statistically significant.

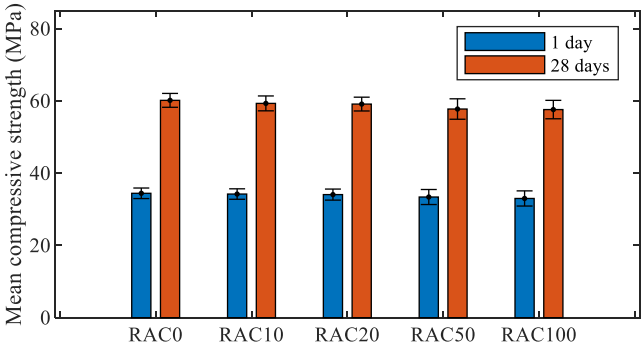


Figure 5-7 – Compressive strength of the different mixtures on one day and 28 days

5.1.3.4 Variability of the compressive strength of RASCC

The maximum and minimum individual compressive strength, the median compressive strength $\widetilde{f_{c,cub}}$, the mean compressive strength $f_{cm,cub}$, the standard deviation s and the coefficient of variation CoV of the compressive strength of

all the tested specimens are summarised in Table 5-7. For each mixture, one batch with 20 cubes is produced and tested on 28 days.

Table 5-7 – RASCC – lab tests – statistical parameters

	$f_{c,max}$ (MPa)	$f_{c,min}$ (MPa)	$\widetilde{f_{c,cub}}$ (MPa)	$f_{cm,cub}$ (MPa)	s (MPa)	CoV (%)	$f_{cm,cub}/f_{cm,cub,RASCC0}$
RASCC0	77.5	67.8	71.0	71.2	2.3	3.3	1.00
RASCC20	76.4	67.2	69.5	70.2	2.7	3.9	0.99
RASCC100	75.0	62.5	68.1	68.5	3.5	5.1	0.98

A replacement of 20%$_{V/V}$ of the coarse fraction of limestone does not result in a considerable decrease of the compressive strength on 28 days, similar to the results of the RAC mixtures. However, the decrease in compressive strength due to the full incorporation of RCA in the SCC mixture is smaller than the results of the conventional RAC in Table 5-6. Section 3.2.1 and 3.2.2 explain that the coarse fraction of limestone, replaced by RCA, is only 27%$_{V/V}$ of the total aggregate skeleton for SCC, while this is 45%$_{V/V}$ for conventional concrete. Therefore, a smaller decrease is expected as a smaller volume of natural aggregates is replaced by RCA.

The effect of a complete replacement of coarse limestone aggregates by RCA on the within-batch variability is clear. The standard deviation and CoV both slightly increase for the mixture RASCC20, and both increase more for the mixture RASCC100. Following statistical analysis will determine whether the small differences are statistically significant and whether this should be taken into account in the design process of precast structural concrete.

5.1.3.5 Evaluation of the variability of the compressive strength of concrete

ACI 214R-11 [103] advises using the CoV-value when the compressive strength is higher than 35 MPa. This standard proposes a system to classify the variability of concrete with natural aggregates. The overall variation and within-batch variation are classified into different categories according to the CoV-

value of concrete's compressive strength, as shown in Table 5-8. This evaluation of the compressive strength variability has been set up based on the compressive strength data analysis by ACI Committee 214. The overall variability of the compressive strength of the mixtures made in the lab in Table 5-6 can be classified as Very Good to Excellent for RAC0, RAC10 and RAC20, while that of RAC50 and RAC100 can be classified as Good to Very Good. The within-batch variability of the compressive strength of RAC0, RAC10 and RAC20 is evaluated as Good to Very Good, while RAC50 and RAC100 are Poor to Good, depending on the batch.

JSCE [81] proposes a threshold limit of 5% for the CoV values of the compressive strength. The CoV values of the compressive strength of the mixtures RAC0, RAC10 and RAC20 are below this value for the 1-day compressive strength, while it is for the mixture of RAC50 and RAC100 above the threshold. For the 28-days compressive strength, all mixtures fulfil this requirement.

Table 5-8 – Classification of the variability of the compressive strength of concrete

Overall Variation					
Class of operation	CoV-value (%)				
	Excellent	Very good	Good	Fair	Poor
Laboratory trial batches	Below 3.5	3.5 to 4.5	4.5 to 5.0	5.0 to 7.0	Above 7.0
Construction testing	Below 7.0	7.0 to 9.0	9.0 to 11.0	11.0 to 14.0	Above 14.0
Within-batch variation					
Class of operation	CoV-value (%)				
	Excellent	Very good	Good	Fair	Poor
Laboratory trial batches	Below 2.0	2.0 to 3.0	3.0 to 4.0	4.0 to 5.0	Above 5.0
Construction testing	Below 3.0	3.0 to 4.0	4.0 to 5.0	5.0 to 6.0	Above 6.0

Based on a study of Rusch [116], European standards [82] conclude that the standard deviation can be used for all strength classes. Rusch et al. [116] proposed an appropriate standard deviation of a range of strength classes under different curing conditions. The author concluded a valid standard deviation of 4.86 MPa, adapted by Eurocode 2 [40] and fib Model Code 2010 [41]. A few years later, Rackwitz [117] proposed a classification system of different standard

deviations, where a standard deviation of compressive strength of field control testing of 4 MPa can be categorized as very good control, 5 MPa as good control, 8 MPa as acceptable and 9 MPa as insufficient quality control. Ostlund concluded in CEB Bulletin 191 [118] an acceptable standard variation of the compressive strength between 4.8 and 6.1 MPa for a range in the compressive strength of 33-69 MPa, based on various tests data from a Swedish ready-mix production of concrete. The CoV of the tests on the compressive strength was 8% - 14%, decreasing with a higher compressive strength.

5.2 Statistical analysis

The results in Section 5.1 show slight differences between the mean values of the compressive strength and its variability between the different concrete mixtures. A statistical analysis is necessary to determine whether these differences are statistically significant. The evaluation of the discrepancies between the different mixtures covers all tests' mean values since the overall mean values and variability of the concrete compressive strength are evaluated in this section.

5.2.1 Chi-square test

The Chi-Square test for Normality [119,120] is used to see whether a normal distribution can be used to fit the distribution of the test data. Given the mean value and standard deviation, the expected values are calculated under the normal distribution for each data point. The Chi-Square Test is defined for the null hypothesis H0: the data follows a normal distribution. The data are divided into k intervals, and the test statistic is defined as in Eq.(5-2).

$$\chi^2 = \sum_{i=1}^{k} \frac{(O_i - E_i)^2}{E_i} \qquad (5\text{-}2)$$

O_i is the observed frequency for interval i and E_i is the expected frequency for interval i. The expected frequency is calculated by Eq.(5-3).

$$E_i = N(F(Y_u) - F(Y_l)) \quad (5\text{-}3)$$

N is the sample size, F is the cumulative distribution function for the tested data, Y_u is the upper limit for class i and Y_l is the lower limit for class i.

The statistical analysis follows a chi-square distribution with (k – 1 – c) degrees of freedom where k is the number of intervals that are not empty and c is the number of estimated parameters. Therefore, the hypothesis that the data are from a population with the specified distribution is rejected if Eq.(5-4) is fulfilled.

$$\chi^2 > \chi^2_{1-\alpha,k-1-c} \quad (5\text{-}4)$$

where $\chi^2_{1-\alpha,k-1-c}$ is the chi-square critical value with k – 1 – c degrees of freedom and significance level α (one-sided).

Table 5-9 shows the chi-square test results for the RAC mixtures. The test data from all the mixtures are normally distributed, as all the χ^2-statistic are below the χ^2-critical (at 0.05 level).

Table 5-9 – Chi-square test results for normal distribution

Testing age	Mixture	χ^2-statistic (calculated)	χ^2-critical (at 0.05 level)	P-value	Accept or reject H_0
One day	RAC0	5.39	7.81	0.15	Accept
	RAC10	3.67	7.87	0.30	Accept
	RAC20	7.24	9.49	0.12	Accept
	RAC50	11.51	12.59	0.07	Accept
	RAC100	10.96	14.07	0.14	Accept
28 days	RAC0	9.36	11.07	0.10	Accept
	RAC10	14.19	16.92	0.12	Accept
	RAC20	8.42	12.59	0.21	Accept
	RAC50	10.13	18.31	0.43	Accept
	RAC100	7.43	16.92	0.59	Accept

Table 5-10 shows the results of the Chi-Square test of the RASCC mixtures, where all test data can fit a normal distribution.

Table 5-10 – Chi-square test results for normal distribution

Mixture	χ^2-statistic (calculated)	χ^2-critical (at 0.05 level)	P-value	Accept or reject H_0
RASCC0	9.10	12.59	0.33	Accept
RASCC20	8.08	12.59	0.43	Accept
RASCC100	6.31	16.92	0.71	Accept

5.2.2 Probability density functions

Probability density functions create a better insight into the mixtures' compressive strength variability. The probability density functions f(x) for the compressive strength of the RAC mixtures are shown in Figure 5-8 and Figure 5-9 for curing ages of 1 and 28 days, respectively. The normal distribution, confirmed by the Chi-Squared tests, is for all mixtures consistent with the histograms. The RAC mixtures with 0%, 10% and 20% replacement have a similar standard deviation and mean values, and the probability density

functions are similar. The RAC mixtures with 50% and 100% replacement have a larger standard variation and lower mean value, similar to Henry et al.'s results [121]. The difference between RAC50 and RAC100 is small for the tests at one day and 28 days.

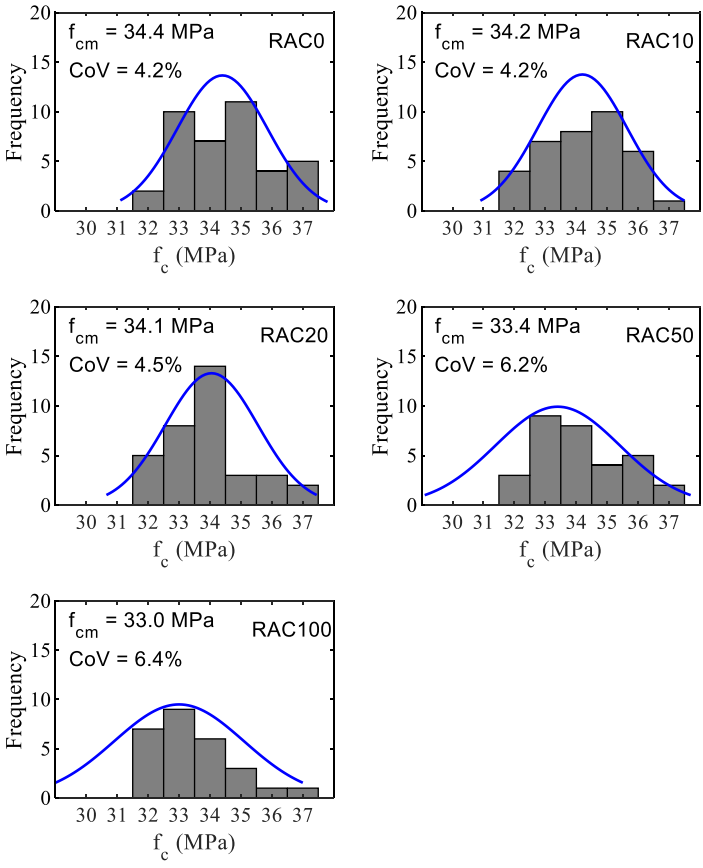


Figure 5-8 – Histogram and distribution density functions for the 1-day compressive strength

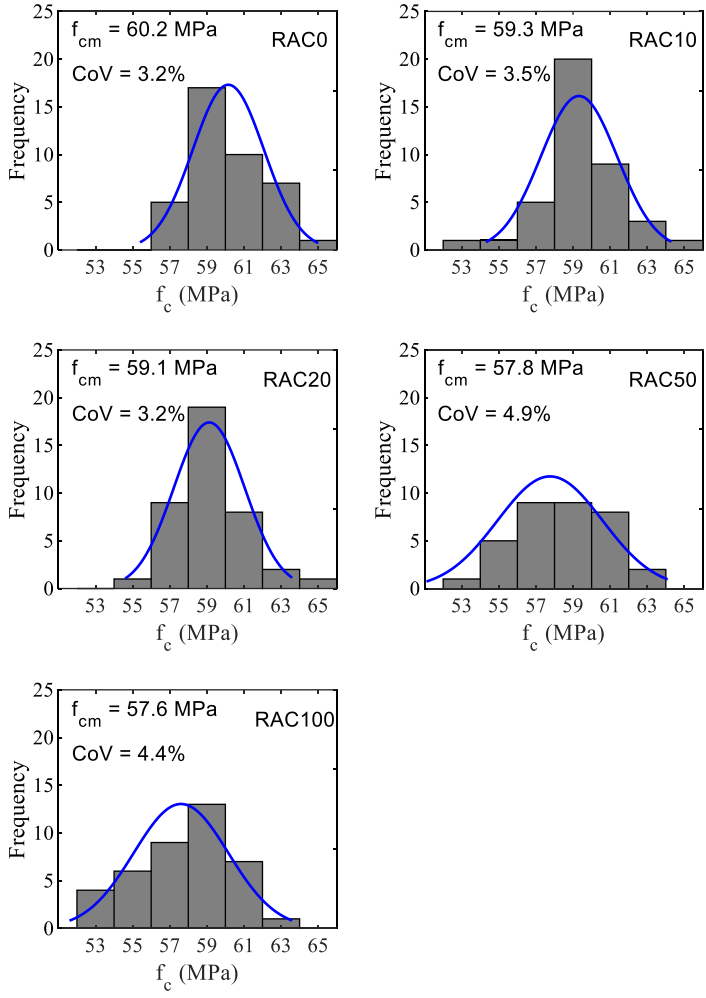


Figure 5-9 - Histogram and distribution density functions for the 28-days compressive strength

The histograms and probability density function of the different datasets of the RASCC-mixtures are shown in Table 5-10. The probability density function widens when larger replacement percentages are used. This is the result of a higher standard deviation. The agreement from the dataset with a normal distribution is confirmed in the histograms' resemblance with the probability density function.

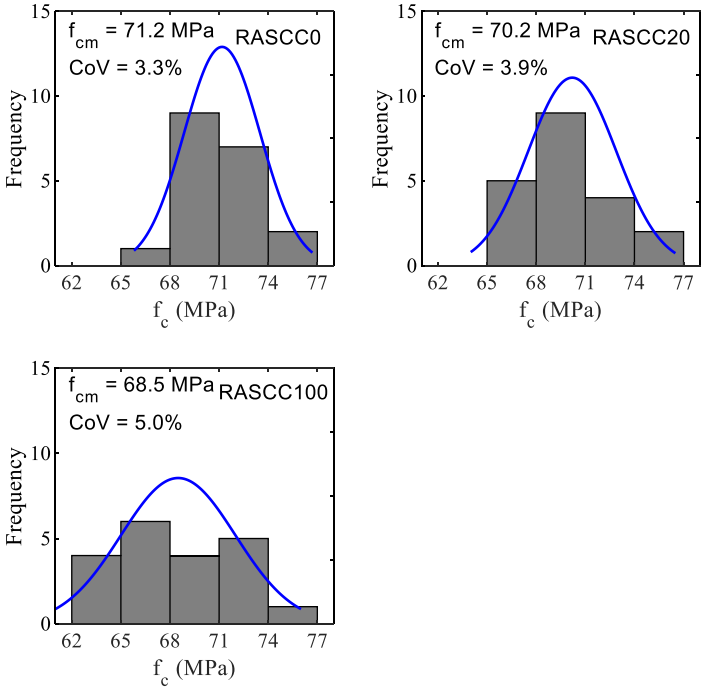


Figure 5-10 – Histogram and distribution density function for the compressive strength of SCC with RCA

5.2.3 F-test

The F-test is performed to investigate any difference in the compressive strength's variability between the mixtures with natural aggregates and the mixtures with RCA [119,120]. The results in Section 5.1 already show some differences, but it is clear that the discrepancies between the variability of the compressive strength of the different mixtures are relatively small. In the F-test,

the variances of the dataset on compressive strength of different mixtures are compared with each other to evaluate any statistically significant differences [119,120]. This is done by calculating the ratio of the two variances, as in Eq.(5-5); s_1 and s_2 representing the sample variances.

$$F = \frac{s_1^2}{s_2^2} \tag{5-5}$$

The F null hypothesis test is defined in Eq.(5-6).

$$H_0: \quad s_1^2 = s_2^2 \tag{5-6}$$

The null hypothesis is rejected if the F-value, resulting from the comparison between two mixtures, exceeds the critical value from the one-tailed F-test, as shown in Eq.(5-7). If so, the variances of the two mixtures are statistically significantly different.

$$F > F_{0.05,N_1\text{-}1,N_2\text{-}1} \tag{5-7}$$

where $F_{0.05,N_1\text{-}1,N_2\text{-}1}$ is the critical value of the F distribution with N_1-1 and N_2-1 degrees of freedom and a significance level of 0.05.

The results are reported in Table 5-11 and Table 5-12. The results from the compressive strength datasets on one day and 28 days are consistent with each other. It is clear that, within the scope of this research, a replacement up to $20\%_{V/V}$ is possible without significantly increasing the variability of the compressive strength. However, the variation of the compressive strength of RAC50 and RAC100 is significantly higher than that of RAC0. This is in accordance with [104] [105] [109], where a replacement percentage of 100% resulted in a significant higher variation of the compressive strength. The reason for the higher variability for RAC can be attributed to an uneven distribution of the amount of attached mortar on the RCA or slight changes in the mix proportions.

Table 5-11 – F-test results of compressive strength at one day

Comparison between different variances	F-value (calculated)	F-critical (at 0.05 level)	Accept or refuse H_0
s_{RAC0}-s_{RAC10}	1.02	1.74	Accept
s_{RAC0}-s_{RAC20}	1.05	1.73	Accept
s_{RAC0}-s_{RAC50}	1.89	1.73	Reject
s_{RAC0}-s_{RAC100}	2.07	1.73	Reject

Table 5-12 - F-test results of compressive strength on 28 days

Comparison between different variances	F-value (calculated)	F-critical (at 0.05 level)	Accept or reject H_0
s_{RAC0}-s_{RAC10}	1.15	1.70	Accept
s_{RAC0}-s_{RAC20}	1.14	1.70	Accept
s_{RAC0}-s_{RAC50}	2.17	1.70	Reject
s_{RAC0}-s_{RAC100}	1.76	1.70	Reject

Table 5-13 shows the F-test results on the SCC with RCA. It can be seen that the larger standard deviation of RASCC100 compared to RASCC0 can be considered statistically significantly larger. The variation on the test data of the compressive strength of RASCC0 and RASCC20 is similar. It can be concluded, similar to the RAC mixtures, that the RASCC mixtures can have a replacement percentage up to 20% without significantly increasing the variability of the compressive strength.

Table 5-13 - F-test results of compressive strength on SCC with RCA

Comparison between different variances	F-value (calculated)	F-critical (at 0.05 level)	Accept or reject H_0
s_{RASCC0}-$s_{RASCC20}$	1.28	2.12	Accept
s_{RASCC0}-$s_{RASCC100}$	2.21	2.12	Reject

5.2.4 ANOVA & Tukey test

The One Way ANOVA [119,120] compares two mean values from two groups of samples. The level of significance used in this one-tailed test is 5%. The null hypothesis for the test is that the different mean values are equal. Therefore, a significant result means that the different mean values are unequal. The one-way ANOVA will not reveal which groups are different if more than two groups are compared to each other. Therefore, if the ANOVA test returns a significant result, an ad hoc test must be run to distinguish which groups had an actual difference in mean values. The Tukey test [119,120] is a one-tailed ad hoc test to determine which mean values of the different groups are statistically significantly lower than that of the reference concrete. The different groups will be directly compared to each other, one by one. The null hypothesis H0 of the Tukey test is that the two mean values of the groups can be considered statistically equal.

There was a statistically significant difference between the mean compressive strength of the different RAC mixtures on one day ($F(4,178) = 4.40$, $p = 0.002$) and on 28 days ($F(4,195) = 9.31$, $p = 0.000001$) as determined by the one-way ANOVA. More details about the ANOVA-tests can be found in Table 5-14.

The Tukey test results for the 1-day compressive strength are shown in Table 5-15. The H_0 is rejected when comparing the mean values of RAC0 and RAC100, indicating no significant difference between the mean values of the compressive strength of RAC0 on one side and RAC10, RAC20 and RAC50 on the other side. The only actual difference is between the mean values of RAC0 and RAC100. The Tukey test results for the 28 days compressive strength are shown in Table 5-16. There is no difference between the compressive strength's mean values of RAC0, RAC10 and RAC20, but a replacement of $50\%_{V/V}$ and $100\%_{V/V}$ results in a significant difference relative to the reference concrete. This is in accordance with [109]. From these results can be concluded that the compressive strength at one day is less affected by the presence of RCA in the concrete than the compressive strength at 28 days. The mortar fraction of the concrete is still weak one day after the concrete production. This would mean the fracture at failure will be intergranular instead of transgranular fractures occurring at later

curing ages. The RCA have less influence on the compressive strength when an intergranular fracture occurs.

Table 5-14 - One way ANOVA: 1-day compressive strength and 28 days compressive strength

1-day compressive strength						
Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	52.40	4.00	13.10	4.40	0.002	2.42
Within Groups	529.43	178.00	2.97			
Total	581.83	182.00				
28-days compressive strength						
Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	193.76	4.00	48.44	9.31	0.000001	2.42
Within Groups	1014.64	195.00	5.20			
Total	1208.40	199.00				

Table 5-15 - Tukey test results on 1-day compressive strength

Comparison between mixtures	T-value	Difference between mean values	Accept or reject H_0
RAC0 - RAC10	1.12	0.21	Accept
RAC0 - RAC20	1.12	0.38	Accept
RAC0 - RAC50	1.12	1.01	Accept
RAC0 - RAC100	1.12	1.44	Reject

Table 5-16 - Tukey test results on 28-days compressive strength

Comparison between mixtures	T-value	Difference between mean values	Accept or reject H_0
RAC0 - RAC10	1.41	0.82	Accept
RAC0 - RAC20	1.41	1.05	Accept
RAC0 - RAC50	1.41	2.51	Reject
RAC0 - RAC100	1.41	2.49	Reject

The ANOVA test also revealed statistical differences between the mean values of the different RASCC mixtures' compressive strength ($F(2,57) = 4.40$, $p=0.017$). More details about the ANOVA tests are found in Table 5-17.

Table 5-17 - One way ANOVA: 28-days compressive strength of SCC with RCA

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	72.4	2	36.2	4.4	0.017	3.16
Within Groups	471.1	57	8.27			
Total	543.6	59				

Table 5-18 shows the results of the Tukey test. The null hypothesis is defined as two mean values that are equal and rejected when the difference in the mean value of the compressive strength of SCC between the mixture is larger than the calculated T-value. The Tukey tests show that the difference in compressive strength between RASCC0 and RASCC100 is statistically significant.

Table 5-18 - Tukey test results on 28-days compressive strength of SCC with RCA

Comparison between mixtures	T-value	Difference between mean values	Accept or reject H_0
RASCC0 – RASCC20	2.51	0.98	Accept
RASCC0 – RASCC100	2.51	2.66	Reject

5.3 Design considerations concerning the variability of compressive strength for precast RAC

This section discusses the effect of RCA on the general principles and criteria used when implementing RCA in structural codes. It is not intended to discuss the direct implementation of RCA into a particular design code.

5.3.1 Reliability of a structure

A structures' safety can be simplified by expressing that the structural resistance R should be larger than the load effect E on the structure [122]. The resistance R and the load effect E are random variables, which do not always have the same fixed outcome. The quantity of loads and material properties are random in which perfect knowledge cannot be achieved. Since these parameters are uncertain, there is a probability that structures might fail, even with correct design and construction [123]. Therefore, these uncertainties must be considered using probabilistic evaluations of different parameters. The possibility of a hazard where a low resistance is combined with a very high load should be kept as low as possible. The failure probability P_f is defined as the probability that the load effects of a structure or structural element are larger than the structure's resistance [122]. P_f can easily be calculated in small members with simple distribution functions. However, multiple random values influence the structure in most practical cases, which complicates the probabilistic evaluation. Therefore, reliability methods are applied to analyse more complex systems. This is translated into instructions and design prescriptions to help the designers in practice.

The uncertainties in structures can be caused by variability in materials properties, environmental actions, human actions and modelling. A better understanding of structural uncertainty can be achieved by considering the uncertainty as a function of three factors [124]:

- Physical factors related to the random nature of materials and environments, such as the difference in concrete composition due to batching, improper mixing and slight discrepancies in curing conditions or reinforcement placement.
- Statistical factors resulting from a lack of perfect statistical information about concrete properties due to a limited number of samples.
- Model uncertainty factors result from designing constructions with theoretical models. In most cases, the practical situation differs from theoretical assumptions, influencing the mathematical models of constitutive relationships between the concrete properties.

Based on these three factors, it is clear that the variability of concrete with RCA should be evaluated differently than conventional concrete. The RCA variability is expected to be higher because of an uneven distribution of attached mortar on the aggregates. Concrete with only natural aggregates has been tested more than RAC, resulting in a higher statistical uncertainty for RAC. Also, different constitutive models are affected by RCA's presence in the concrete.

5.3.2 Reliability of precast RAC

Each step in the production process of concrete includes the potential for variability. The key is to control as many factors as possible to ensure consistent concrete quality. Concrete will always have an inevitable variability, but this should be minimised by controlling as many variables as possible.

In research studies, concrete variability is mainly tested in lab controlled conditions, which is the most favourable condition with the slightest variations in compaction, curing, temperature... When the concrete is taken into mass production, many variables arise that can influence the final concrete product's variability.

There are several types of concrete production types with different expected variability levels [22]. They are discussed in a decreasing level of variability:

- Concrete made and cast on the construction site is expected to have the highest mechanical properties variability. The proportioning of raw materials, mixing, and concrete placement is subjected to many variables that can influence the variability of the concrete's final mechanical properties. This type of concrete production is usually not recommended when consistent quality is required.
- Ready-mixed concrete is made in a controlled environment like a concrete plant but is then transported to the construction site in a fresh state, where it is poured and compacted. The proportions and mixing process are controlled to minimise any variability between mixtures, but the placement on the construction site depends on human workmanship.

- Precast concrete has a production process with the slightest chance of factors influencing the variability. Generally speaking, precast concrete can be designed with a lower variability compared to ready-mixed concrete, which is on its turn generally more reliable than concrete produced on site.

5.3.3 Statistical and probabilistic studies on RAC properties

As the previous tests and analysis in Sections 5.1 and 5.2 reveal, the use of high-quality RCA directly influences the variability of the precast concrete mixtures.

The within-batch variability of concrete properties can be affected by RCA's presence. Intermediate RCA ratios and complete replacement of RCA can increase mechanical properties' variability [110]. The analysis mentioned in Section 5.2 concludes a statistically significant increase in the within-batch variability when high-quality RCA is used in precast structural concrete. Small discrepancies in mix proportions between different test samples and an uneven distribution of attached mortar among the RCA results in a change in local W/C-ratio. This logically increases the variability of the properties of RAC. Table 5-5 and Table 5-6 show that the CoV-value increases when the coarse fraction of limestone is replaced by RCA.

The variability of concrete properties among different batches can also be affected by RCA's presence. RCA's quality and properties can slightly change among different batches, which will affect the final properties of concrete with RCA. Small changes in the water absorption of the used RCA in different batches will affect the effective W/C ratio of the mixture, increasing the concrete variability between batches. An overestimation of the WA_{24} will increase the free water and the W/C ratio, while an underestimation of the WA_{24} will lead to a lower W/C ratio because the RCA will absorb more water than foreseen. From Figure 5-6 in Section 5.1 can be concluded that the influence of RCA on the batch-to-batch variability of the compressive strength of concrete is limited due to a relatively constant quality of the used RCA. It should be noted that this conclusion is based on the fact that the RCA in

different batches originated from the same type of concrete waste by the same recycling plant. RCA from different sources might increase the batch-to-batch variability of concrete.

Special attention is given to the test design of this research in terms of the sample number since an essential parameter in the realistic evaluation of the variability of concrete is the sample size. In most studies, a rather limited amount of test samples is used. This can lead to inaccurate representations of the variability of concrete with RCA and hinder code calibration. A large number of samples is necessary to adequately describe the material properties' probabilistic models involved in structural verifications [123]. With a limited amount of tests, most data is concentrated around the mean values of the tests, while more information on the lower part of the probability curve is necessary for a reliable calculation of the characteristic values of the concrete properties.

Xiao et al. [108], Etxeberria et al. [109], Henry et al. [105], and Breccolotti and Materazzi [110] have reported extensive studies with a large number of samples. Table 5-19 summarises the statistical evaluation of the data of these sources. Except for [20], all sources use industrially produced RCA. Xiao et al. [108] concluded that the variability increases when different types of coarse aggregates are mixed. The use of intermediate replacement percentages of RCA subsequently increases the concrete compressive strength variability compared to $0\%_{V/V}$ or $100\%_{V/V}$ replacement. Breccolotti and Materazzi [110] tested the effect of different RCA types on the within-batch variability of concrete's compressive strength. The results showed no clear tendency on the influence of RCA. Etxeberria et al. [109] and Henry et al. [105] reported higher variability when RCA fully replace the coarse fraction. These conclusions are similar to that in Section 5.2; however, it should be noted that a complete replacement of the coarse limestone fraction in the present research only equals a replacement percentage of 90% of the whole coarse fraction.

The statistical results of the compressive strength tests mentioned in Table 5-19 can be compared with the different standards' thresholds. As already discussed in Section 5.1.3.5, Eurocode 2 [40] and fib Model Code 2010 [41] prescribed a valid standard deviation of 4.86 MPa. Rackwitz [117] suggested a standard

deviation of 5 MPa to be a good value, while 8 MPa was still acceptable. Ostlund concluded in CEB Bulletin 191 [118] an acceptable standard variation of the compressive strength between 4.8 and 6.1 MPa for a range in compressive strength of 33-69 MPa. Most results in Table 5-19 turn out to be below the above sources' thresholds. However, it should be noted that all data in Table 5-19 were derived from laboratory tests, controlling as many parameters as possible. The actual variability in the practical situation will most likely be higher. In this regard, it is necessary to develop or use models transferring the test data of concrete with RCA obtained from laboratory environments to practical situations [125].

ACI 214R-11 [103] does make a difference between laboratory trial batches and general construction testing. Table 5-8 already showed a difference in classifying the variability of the concrete compressive strength made in a lab environment and on the construction site. The results of Xiao et al. [108] and Etxeberria et al. [109] can be classified as poor, both for concrete with natural aggregates and concrete with RCA. Other results can be classified as Fair and Good, depending on the mixture. Henry et al. [105] evaluated RCA's effect on the within-batch variability of concrete. The variability of concrete compressive strength with natural aggregates can be classified as Very good, while the variability of the compressive strength of concrete with RCA can be classified as Good or Fair.

Table 5-19 – Statistical results of studies of the 28-day compressive strength of concrete with RCA (extension of the table in [123])

	RCA (%)	No. of samples	Mean f_c (MPa)	St.Dev f_c (MPa)	CoV f_c (%)	CoV_{RAC} / CoV_{NAC}
Present study	0	40	60.2	1.9	3.2	1.00
	50	40	57.8	2.8	4.9	1.53
	100	40	57.6	2.6	4.4	1.38
	0	20	71.2	2.3	3.3	1.00
	20	20	70.2	2.7	3.9	1.18
	100	20	68.5	3.5	5.1	1.55
Xiao et al. [108]	0	75	41.6	3.4	8.3	1.00
	30	100	41.5	3.9	9.5	1.14
	50	98	40.2	3.9	9.7	1.17

	100	98	36.5	3.0	8.2	0.99
Etxeberria et al. [109]	0	24	44	3.9	8.8	1.00
	25	15	43	4.0	9.3	1.06
	50	20	46	4.3	9.3	1.06
	100	15	46.5	5.8	12.6	1.43
Henry et al. [105]	0	30	41.9	1.2	2.9	1.00
	100	30	34.4	1.3	3.9	1.34
	100	30	32.7	1.3	4.3	1.48
	100	30	28.8	1.2	4.4	1.52
Breccolotti and Materazzi [110]	0	10	50.3	3.2	6.4	1.00
	50	10	54.9	1.5	2.8	0.44
	100	10	52.3	1.2	2.3	0.36
	0	10	50.7	1.5	3.0	1.00
	50	10	45.4	1.6	3.5	1.17
	100	10	53.2	1.7	4.6	1.53
	0	10	37.8	1.9	5.0	1.00
	50	10	43.9	1.7	3.8	0.76
	100	10	38.3	1.8	2.5	0.50
	0	20	49.2	3.7	7.4	1.00
	50	20	33.7	3.3	9.6	1.30
	100	8	53.2	3.4	10.8	1.46

To conclude, the variability of the compressive strength of concrete with RCA has been studied to some extent, but more studies are still necessary to justify the given conclusions. The limited number of studies conclude an increased CoV-value due to RCA replacement up to 50%. However, this increase seems acceptable when comparing the standard deviation and CoV value of the mixtures with different threshold values from different standards. Besides, the compressive strength is the only known property where RCA's influence on the variability is examined, except for the research reported by Xiao et al. [126], where the variability of the splitting tensile strength of concrete with RCA was also studied. The results showed higher variability in the splitting tensile strength than the compressive strength, which is also the case for concrete with natural aggregates. More studies on the variability of tensile strength, elastic modulus and other mechanical and durability properties are necessary to clarify the effect of RCA on the concrete properties.

5.3.4 Reliability of RAC elements

A deeper understanding of the structural reliability of RAC is necessary, even though the studies on structural concrete with RCA, described in Section 1.3, show a similar behaviour between structural concrete with RCA and natural aggregates. The effect of an increased heterogeneity of the RCA on the reliability of the structural behaviour of concrete has been investigated in several studies using the reliability index β [102,106,110,127]. The reliability index β is used as an alternative method to estimate failure probability. In the specific case of concrete structures, the reliability is defined by the difference between the concrete's resistance (R) and the effective forces or loads upon the concrete (E). The failure probability is the chance that the R-E is smaller than 0.

The calculation of P_f is burdensome, and simple calculation methods may not exist in some cases. Above that, a full calculation of P_f is often not possible because of insufficiently precise probabilistic information. The reliability index (β) is an estimation of reliability, providing an alternative for the full calculation of P_f. Eq.(5-8) is a simplified calculation of the reliability index, illustrating the idea of β. An increase of the standard deviations of the variables influencing R and S will lower the reliability, while a significant difference between the expected value of the resistance effect and the load effects will result in a high reliability.

$$\beta = \frac{(\mu_R - \mu_S)}{\sqrt{(\sigma_R^2 + \sigma_S^2)}} \qquad (5\text{-}8)$$

Breccolotti and Materazzi [110] evaluated the influence of the higher variability of RAC compressive strength on structural reliability. This study's statistical parameters are experimentally obtained and discussed in Section 5.3.3. Based on the experimental results, the safety coefficient has been calculated to secure structural concrete beams with RCA made of the same failure probability as concrete beams with natural aggregates. The β-index has been evaluated with the procedure proposed by Rackwitz and Flessler [128]. When axial-loaded columns' safety is evaluated, the partial safety factor γ_c increases up to 2.1. A

higher percentage of RCA results in a higher γ_c. RCA's effect on γ_c increases when the quality of the RCA decreases.

Breccolotti and Materazzi [106] also performed a theoretical study to assess the reliability of the bond between deformed reinforcement bars and structural concrete with RCA. The authors evaluate the necessity to change the design safety factors to create a similar P_f as NAC. A reference CoV-value of 15% was assumed for NAC, while a higher scattering of RAC was translated into a 30% CoV-value. The results show that the partial safety coefficient slightly increases with an increase in the CoV-value of the bond between reinforcement and concrete. The authors state that the safety coefficient does not need an increase for the bond between reinforcement and RAC since the standards generally provide a conservative approach towards structural reliability.

Xiao et al. [127] proposed a method in which the reliability of concrete RAC can be enhanced by increasing the reinforcement ratio. The analysis starts with the assumption that NAC and RAC have a similar mean compressive strength by adjusting the W/C ratio of RAC. The CoV-value of the compressive strength of RAC is assumed to be maximal 20%. The load effects are kept constant to NAC beams. The RAC beam's reliability index can be obtained using the method proposed by Rackwitz and Flessler [128]. The results show that the RAC beam's reliability index decreases as the CoV-value of RAC increases. The reliability of the beams with RAC can increase by adjusting the reinforcement ratio or improving mechanical properties. The results show that the maximum reinforcement ratio of beams with RAC should be increased by 3.5% to ensure a similar reliability index as beams with NAC.

The different studies on the structural reliability of RAC have adopted different approaches. It should be noted that these studies have some limitations. The reliability analysis is based on estimations of the probabilistic information of RAC or based on laboratory studies with a limited amount of specimens. Also, only the increased variability of the compressive strength of concrete with RCA has been included in the reliability analysis.

5.3.5 Partial safety factors

The actual strength of concrete in a structure is often smaller than characteristic strength because of the workmanship or quality control in the manufacturing process, especially when the concrete is poured on the construction site itself. The uncertainties due to the conversion from the conventional concrete strength determined using control specimens to the actual strength in the concrete structure are considered and included in the partial safety factor for concrete γ_c. The final design strength is obtained by considering the partial safety factor into the characteristic strength.

The design strength of the material f_{cd} is given by Eq.(5-9).

$$f_{cd}=\alpha_{cc}\frac{f_{ck}}{\gamma_c} \tag{5-9}$$

With α_{cc} being a coefficient considering long-term effects on the compressive strength and unfavourable effect of the way of loading.

New materials in practical situations require particular guidelines in codes or standards, which is essential for all applications where concrete with RCA is used. The properties of concrete with RCA are related to the structural resistance R of a concrete element. The higher variability of the resistance R will result in lower reliability. The partial safety factors designed for NAC may not be suitable for the use of RCA in concrete, as the target P_f may not be reached for concrete structures with RCA. They should be calibrated so that designers can use RAC without any restraint based on doubts about its quality.

Eurocode 2 specifies the partial safety factors for materials. The partial safety factor γ_c for concrete in ULS is 1.5 [1]. A lower value for γ_c can be used in special conditions mentioned in Annex A. The current version of the Eurocode 2 was published in 2004 and requires a thorough revision to understand the concrete behaviour and the materials currently used in the construction industry. The revision, expected to be published in the coming years, provides a detailed calculation method for the partial safety factors [129]. The current partial safety factor of 1.5 is maintained in general conditions. For precast

concrete, this factor can be reduced to 1.4 if the execution ensures that geometrical deviations of Tolerance Class 2 are fulfilled, and the Assessment and Verification of Constancy of Performance 2+ (AVCP) is applied. A detailed calculation method for γ_c is specified for further adaptions. The adjusted partial factors may be calculated by Eq.(5-10) and Eq.(5-11).

$$\gamma_c = \frac{\exp(\alpha_R \, \beta \, V_{RC})}{\mu_{RC}} \tag{5-10}$$

Where

- α_R = 0.8 = the contribution of the material resistance in the total variability at the design point [129]
- β = 3.8 for ULS based on estimated value EC0 for t_{ref} = 50 years [130]
- V_{RC} = the coefficient of variation of concrete, which is calculated from Eq.(5-11) [129]
- μ_{RC} = the bias factor of the concrete – 1.142 in normal conditions

$$V_{RC} = \sqrt{V_{fc}^2 + V_{\eta is}^2 + V_{Ac}^2 + V_{\vartheta c}^2} \tag{5-11}$$

Where

- V_{fc} = coefficient of variation of the compressive strength
  - 0.10 in normal conditions
- $V_{\eta is}$ = coefficient of variation of the in-situ factor η_{is}
  - 0.12 in normal conditions
- V_{Ac} = coefficient of variation of the concrete area A_c
  - 0.04 in normal conditions
- $V_{\vartheta c}$ = coefficient of variation of the model uncertainty
  - 0.06 in normal conditions

The determination of the factor V_{fc} is in the context of this research highlighted. The general assumed CoV of the compressive strength is 10% [131]. This value is based on existing research results from ready-mixed concrete. These results also confirmed that the CoV value of precast concrete is remarkably lower due to the better-controlled environment of the production. A CoV-value of 6% is

assumed for precast concrete, resulting in a partial safety factor of 1.40, as earlier discussed [131].

The results in Table 5-19 show that the CoV-value increases when RCA are incorporated into concrete. This increase can be up to 50%, both for intermediate and full replacement percentages of the coarse fraction. The absolute value of the CoV of the mixtures' compressive strength in Table 5-19 varies according to the used mixtures. A direct replacement of the assumed CoV of NAC's compressive strength by Eurocode 2 would not be reasonable since this can vary between different mixtures and strength classes. However, a relative increase of 50% relative to NAC would be more convincing and valuable. This would result in a general CoV-value of 15% for RAC. RCA in concrete can also influence some other parameters in Eq.(5-10) and Eq. (5-11). RCA's effect on these parameters should be investigated before implementing a possible adaption of the partial safety factor in structural concrete with RCA.

This conclusion is estimated based on the available statistical studies on RCA use in concrete. More studies are necessary to confirm these results. It should also be noted that the above reasoning is based on the provisional calculation of the partial safety factor in the draft version of the new revision of Eurocode 2. Final conclusions can be made when the new revision of Eurocode 2 is published.

5.4 Conclusions

The effect of the high-quality RCA on the variability in concrete compressive strength has been measured thoroughly. The statistical analysis is based on experimental results of precast structural concrete with the high-quality RCA. Following conclusions can be made within the scope of this study.

Due to the attached mortar, the RCA has a higher WA_{24} and a lower ρ_{RD} than natural aggregates. The WA_{24} and ρ_{RD} of the RCA show a higher spread than natural aggregates, most likely because of an uneven distribution of attached mortar. The properties of the RCA slightly change during the mixing process.

The WA_{24} of RCA decreases during the mixing process due to the grinding of RCA to the mixer. The sieving curves of the aggregate skeleton after the mixing process show a slightly finer distribution. The standard deviation of the properties of the aggregate skeleton after the mixing process is higher for mixtures incorporating $100\%_{V/V}$ RCA. Small changes in mix proportions, together with the uneven distribution of attached mortar of the RCA, causes a higher variability in the coarse aggregate skeleton's properties, as explained in Section 5.1.3.

A decrease in compressive strength of 4% is observed when the high-quality RCA entirely replace the coarse fraction of limestone in the conventional precast mixture. A replacement percentage of up to $20\%_{V/V}$ does not statistically significant change the compressive strength. The SCC precast mixture showed a smaller decrease in compressive strength when 100% RCA was used due to the smaller percentage of coarse fraction in the aggregate skeleton.

The within-batch variability evaluates the variability in compressive strength of concrete within the same batch. It is tested for the conventional precast mixture and SCC precast mixture. The conventional precast mixture showed an increase in within-batch variability on a curing age of 1 day and 28 days when at least $50\%_{V/V}$ of the coarse fraction of limestone is replaced. The SCC precast mixtures showed no statistically significant difference in within-batch variability of concrete for a replacement percentage up to $20\%_{V/V}$. Also, the incorporation of $100\%_{V/V}$ RCA in the SCC precast mixture results in increased compressive strength variability.

The batch-to-batch variability or repeatability of the concrete quality between batches is tested for the conventional precast mixture. It is observed that the variability of the compressive strength can slightly vary between different batches, both for concrete with natural aggregates or with RCA. There is no clear tendency that high-quality RCA influence the batch-to-batch variability of the compressive strength of the mixtures. It should be noted that the tests were carried out with the high-quality RCA from the same crushing company. A more considerable variation in batch-to-batch variability is expected when different RCA sources are used.

Chapter VI

Bond properties between RAC and steel reinforcement

Chapter based on the publication: Sierens Z., Cai, J., Van Steen C., Verstrynge E., Li J. (2020) Bond performance of deformed steel rebars in HSC incorporating industrially produced recycled concrete aggregates. *Materials and Structures*

The objective of this chapter is to evaluate the local bond behaviour of deformed steel bars in HSC with industrially produced high-quality RCA and to develop a local bond-slip model for use in nonlinear finite element simulations of structural components and structures incorporating this material. To this end, an extensive laboratory study is carried out in this chapter using the high-quality RCA produced from a Belgian recycling plant. The effects of the RCA on the mechanical properties of HSC as well as the local bond-slip between the steel rebars and concrete are investigated.

6.1 State of the art

A large number of laboratory tests on the bond behaviour between steel rebars and concrete containing RCA have been conducted in the past years, for instance, in [45,47,137–146,48,147,62,106,132–136], to examine the effect of RCA on the maximum bond stress and slip behaviour. It is well known that the bond of deformed steel rebar in concrete is mainly transferred by the mechanical bearing of the rebars against the concrete surface, while the chemical adhesion between the rebar and the concrete as well as the frictional forces arising from

the roughness of the interface of the two materials are governing factors for the bond between plain (smooth) steel rebar and concrete. This implies that the effect of RCA can be different when steel rebars of different types are used. In previous studies, both smooth and deformed (ribbed) steel rebars have been included. Different test methods have been used in former studies, and deformation controlled pull-out tests based on the RILEM recommendation [148] were the primary test method. This section examines the bond of deformed steel rebars in HSC with high-quality coarse RCA using pull-out tests. Due to this reason, the following literature survey mainly covers previous work with deformed steel bars embedded in concrete containing coarse RCA subjected to pull-out tests.

Earlier studies mainly focused on the effect of RCA on the bond behaviour between steel rebars and concrete. Ross [132] investigated the bond of deformed steel rebars in concrete with 0 and 100% RCA, respectively. It was reported that the concrete with the RCA exhibits a lower compressive strength, while the ratio of the measured bond strength to the concrete compressive strength is slightly higher when RCA is used. This means that incorporating RCA is beneficial for improving the bond strength of deformed steel rebars in concrete as long as the compressive strength of the concrete mixtures is similar. Later, Ajdukiewicz and Kliszczewicz [133], Corinaldesi and Moriconi [134], Wardeh et al. [135] and Siempu and Pancharthi [45] reported similar results. Xiao and Falkner [136] tested a total of 18 specimens where ribbed steel rebars were embedded in concrete cubes containing RCA produced from the demolished concrete of an airport runway. In their tests, the replacement percentage of the RCA was 0, 50 and 100%, respectively. The test results revealed that although an increase of the RCA decreases the concrete compressive strength, the RCA does not affect the bond strength between the deformed steel rebars and concrete. However, the normalised bond strength, defined as the ratio of the maximum bond stress to the square root of concrete compressive strength, increases with the RCA replacement percentage. Similar findings were also observed by Choi and Kang [137] as well as Breccolotti and Materazzi [106].

Prince and Singh [138,139] carried out an extensive laboratory study on the influence of various test variables on the bond behaviour between deformed steel

rebars and RCA concrete. Their test results implied that the RCA amount does not affect the measured maximum bond stress while the normalised bond strength tends to increase with the RCA replacement percentage, especially for specimens with steel rebars of small diameters. The observations are consistent with the observations summarised above. Kim and Yun [140], Kim et al. [141], and Prince and Singh [142,143] found similar results.

However, Kim et al. [144], Seara-Paz et al. [47] and Eiras-Lopez et al. [62] reported that the incorporation of RCA decreases the maximum bond stress between deformed steel rebars and concrete. It was also found that the normalised bond strength also decreases with the increase of the RCA replacement percentage. Obviously, the test results are not entirely consistent.

Some other factors on the bond of deformed steel rebars in RAC were also studied in previous work. Kim and Yun [140] examined the effect of RCA particle size as well as the direction and location of the steel rebars. Siempu and Pancharthi [45] and Pour and Alam [145] investigated the effect of the rebar diameter, the concrete cover thickness and rebar embedded length on the bond-slip behaviour. The test results indicate that an increase of the rebar diameter and embedded length in concrete leads to a decrease of the maximum bond stress while the increase of the concrete cover thickness enhances the bond strength, both for conventional concrete with natural aggregates and RCA concrete. Those results provide useful information about the bond behaviour of deformed steel rebars in RAC.

Several past studies also discussed how to estimate the maximum bond stress between reinforcement and RAC. It is well known that the bond of steel bars in concrete is a very complex problem. Many factors can affect the maximum bond stress. These influencing factors were summarised in ACI 408 [149], including:

- Structural characteristics, for instance, the concrete cover thickness, embedded length of the rebars, the existence of transverse reinforcement, rebar casting position...;
- Properties of the rebars, which include the size, geometry, yielding strength and surface condition of the rebars;

- Concrete properties, including the compressive strength, aggregate type and quality, tensile strength and fracture energy, admixtures and fibre reinforcement...

An accurate estimation of the maximum bond stress needs to consider all influencing factors. However, this would lead to very complicated equations to predict that parameter. For practical reasons, some simplifications have to be made. Numerous simplified models for calculating the maximum bond stress between steel rebars and concrete were proposed in the literature. A review of existing models can be found in Siempu and Pancharthi [45]. The most frequently used equation for predicting the maximum bond stress $\tau_{b\,max}$ of steel rebars embedded in well-confined concrete (exhibiting pull-out failure) is the one in *fib* Model Code 2010 [82], which is expressed as:

$$\tau_{b\max} = 2.5\sqrt{f_c} \qquad (6\text{-}1)$$

However, many researchers [106,138–141] have reported that Eq.(6-1) underestimates the maximum bond stress for RAC (as well as for concrete with natural aggregate).

Siempu and Pancharthi [45] proposed to use the following equation to estimate the maximum bond stress of deformed steel rebars embedded in concrete:

$$\tau_{b\max} = (k_1 \frac{d_b}{l} + k_2 \frac{c}{l} + k_3)\sqrt{f_c} \qquad (6\text{-}2)$$

in which k_1, k_2 and k_3 are parameters depending on the type of the aggregates. For concrete with natural aggregate: $k_1 = 6.32$, $k_2 = 0.26$ and $k_3 = 0.21$; while for concrete with 100% RCA: $k_1 = 6.38$, $k_2 = 0.44$ and $k_3 = -0.50$ according to Siempu and Pancharthi [45]. d_b is the diameter of the reinforcement bar (mm), c is the distance from the deformed steel rebar surface to the concrete surface (mm), and l is the embedded length of the reinforcement in the concrete samples (mm).

Seara-Paz et al. [47] suggested the following equation to estimate the maximum bond stress based on Eq.(6-1).

$$\tau_{b\max} = 2.5\sqrt{f_c}(1-0.124G) \quad (6\text{-}3)$$

where G is the replacement percentage of the (coarse) RCA to the total coarse aggregate. It should be noted that according to Eq.(6-3), the maximum bond stress reduces with the increase of the RCA content.

A similar equation was proposed by Kim et al. [144] as follows.

$$\tau_{b\max} = 0.614\sqrt{f_c}(\frac{c_c}{d_b} - 0.55) - (0.4203 + 0.7G) \quad (6\text{-}4)$$

where c_c is the distance from the deformed steel rebar centre to the concrete surface (mm). Kim et al. [140] proposed Eq.(6-5) to estimate the maximum bond stress:

$$\tau_{b\max} = k\sqrt{f_c} \quad (6\text{-}5)$$

in which, k is a parameter depending on the rebar diameter: k = 4.4 for d_b = 16 mm while k = 3.9 for d_b = 19 mm.

From existing studies in the literature can be concluded that the bond behaviour between deformed steel rebars and RAC is not yet fully understood. As can be seen from the above literature review, existing test results on the effect of RCA on the maximum bond stress are not consistent, which may be caused by the difference in the properties of the used RCA concrete mix proportions, test methods... Moreover, many previous studies, such as [45,106,133,138,139,142–144], used laboratory-produced RCA, which may not adequately represent RCA produced from the industrial scale. Furthermore, previous research work [106,134–140,144,145] mainly focused on the bond of deformed steel rebars in normal-strength RAC, while relatively little work has been done on the bond of rebars in HSC with RCA. As mentioned above, HSC is frequently required in precast plants to achieve an adequate early-age compressive strength of concrete. Finally, most existing tests adopted an embedded length of the

deformed steel rebars of 5-10 times the rebar diameter, for instance, in [47,106,144,145,134–139,142,143]. Although this is consistent with the RILEM regulations [148], it may be inadequate to investigate local bond behaviour. Based on these facts, the following variables are adopted in this chapter: (1) an industrially produced RCA to facilitate the transfer of the knowledge from the laboratory to practice; (2) a high-strength RAC; (3) a short embedded length, namely three times the rebar diameter to investigate the local bond-slip behaviour between the steel rebar and concrete more realistically.

6.2 Test program

6.2.1 Materials and mix design

The conventional precast concrete mixture used in this experimental campaign is described in Section 3.2.1. The coarse fraction of the aggregates is replaced by several replacement percentages of RCA, 0, 10, 20, 50 and 100%$_{V/V}$, respectively. The influence of RCA on the bond behaviour between concrete and steel reinforcement is tested by the five mixtures shown in Table 3-4. The concrete is produced according to the mixing procedure described in Section 3.3.1 in the Laboratory of Civil Engineering, KU Leuven.

The deformed steel rebars used in the test have a nominal diameter d_b of 16 mm and conform to the Belgian standard NBN EN 10080 [150]. Figure 6-1 shows a test sample of the steel rebars. The minimum yield strength of the rebars is 500 MPa, and the minimum ultimate strength/yield strength is 1.05, with total elongation of 5%. The surface characteristics of the rebars are presented in Table 6-1.

Table 6-1 – Surface characteristics of the rebars

Diameter	16 mm
Rib height	0.96 mm
Rib width	2.0 mm
Rib spacing	8.1 mm
Rib face angle	55°



Figure 6-1 - Deformed steel rebar

6.2.2 Specimen design and preparation

As mentioned above, this study aims to investigate the local bond-slip behaviour of deformed steel bars in HSC with different amounts of RCA and develop a local bond-slip model for use in nonlinear finite element structural simulations. For this purpose, a pull-out test with a relatively short embedded length of the deformed steel rebars is employed. Figure 6-2 presents the test design based on the proposal of Rehm and Eligehausen [151], which represents an improvement of the test method by Rehm [152]. As can be seen from the figure, in the test specimen, a 460 mm-long steel rebar is partially embedded in a concrete block (150 mm × 150 mm × 260 mm). The rebar's effective embedded (bond) length is $3d_b$, which is 48 mm. The concrete cover of the test specimen has a thickness c of $4.2d_b$. The samples used in this study are different from those in the RILEM recommendation [148] in that the present samples have an embedded length of $3d_b$. The shorter embedded length is considered more suitable for measuring the local bond-slip behaviour since it permits us to assume a uniform distribution of the bond stress along with the rebar [153]. Outside the effective bond zone, the bond of the steel rebar to the concrete is deactivated using 2 PVC tubes, as illustrated in Figure 6-2. The load is applied on the steel rebar protruding at the bottom of the concrete block. The loaded end of the rebar has a length of 150 mm to ensure a good connection between the rebar and the clamp of the machine.

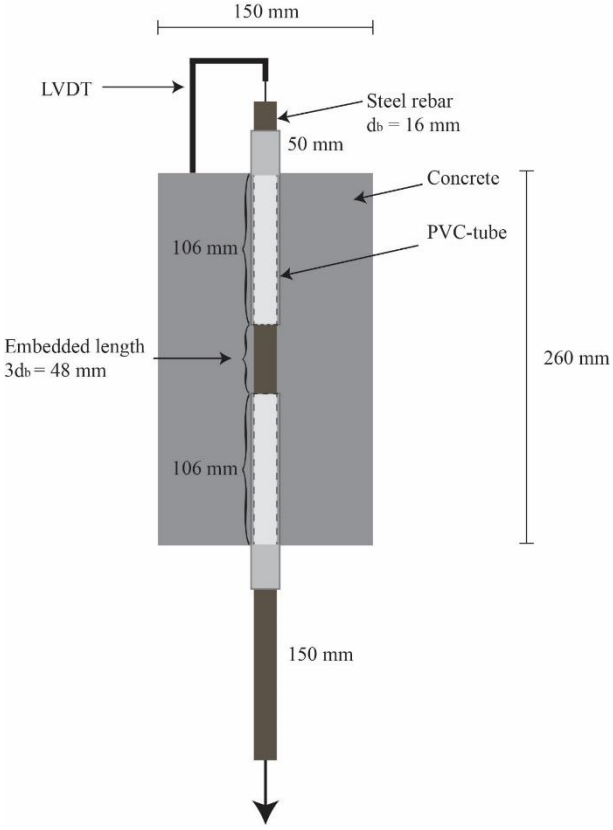


Figure 6-2 - Sketch of the pull-out specimen

For each concrete mixture, a total of three cube specimens (150 mm × 150 mm × 150 mm) and six prism specimens (150 mm × 150 mm × 260 mm) are cast. The former is used to measure the compressive strength of the concrete, while the latter is used for the pull-out test. The pull-out specimens are cast with the steel rebars in a horizontal position to simulate the anchorage of end rebars in reinforced concrete beams. Immediately after the mixing process, a slump test is done for each concrete mixture. After the slump test, the concrete mixtures are cast in wooden moulds and compacted on a vibration table in three layers. After casting each layer, the concrete mixtures are subjected to vibration for 15 seconds. All specimens are demoulded 20 to 24 hours after casting and then cured under standard curing conditions (20°C and 95% RH) in a climate room

until the test. Figure 6-3 shows the moulds for the pull-out specimens as well as some typical pull-out specimens after casting and before the test.



(a) Moulds



(b) After casting



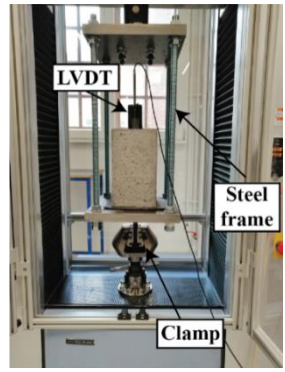
(c) Before test

Figure 6-3 – Preparation of the pull-out specimen

Besides, identical concrete mixtures, as described above, are manufactured in the Laboratory of Construction, KU Leuven Campus Bruges. For each mixture, a total of six cube samples (150 mm x 150 mm x 150 mm) and six cylinder samples (Φ150 mm x 300 mm) are cast. The former is used to measure the concrete splitting tensile strength, while the latter is used to determine the modulus of elasticity and the cylinder compressive strength of the concrete. All those concrete specimens are prepared and cured under the same condition.

6.2.3 Test methods and procedures

All tests, including the tests on the strength properties and modulus of elasticity, as well as the pull-out tests, are performed at a curing age of 28 days. The tests on the compressive strength, splitting tensile strength, and modulus of elasticity are carried out according to [74], [75] and [76], respectively. The pull-out tests are performed using a Shimadzu universal testing machine (maximum load: 100 kN), as shown in Figure 6-4. A loading frame is used to fix the test specimen when the steel rebar is pulled (downwards). The steel rebar is clamped to the test machine in order to apply the pull-out load, see Figure 6-4. A linear variable differential transformer (LVDT) is attached to the free end of the rebar and measures its slip, as can also be seen from Figure 6-4. All pull-out tests are performed under deformation control at a loading rate of 0.3mm/min. The pull-out loads and relative slips (i.e., free end slips) between the steel rebars and concrete are automatically recorded by the data acquisition system of the machine.



(a) Testing machine (with one test specimen)



(b) LVDT measuring the slip of the steel rebar



(c) Clamping of the steel rebar

Figure 6-4 – Test setup

6.3 Test results and analysis

6.3.1 Workability

The measured slump values for the concrete mixtures are presented in Table 6-2. The measured slump value varies from 190 to 200 mm, indicating that all concrete mixtures reach the consistency class of S4 in EN 206 [12]. The measured slump value of the RAC is comparable to that of the reference concrete, which is attributed to the addition of the compensation water.

6.3.2 Strength properties

The measured cube compressive strength $f_{c,cub}$, cylinder compressive strength $f_{c,cyl}$ and tensile splitting strength $f_{ct,sp}$ of the concrete mixtures are given in Table 6-2. The values in the table represent the mean values and standard deviation of the data measured on three or six test specimens. The standard deviations of the measured strength properties are also presented in the table. It can be observed from the table that both the cube and cylinder compressive strength of the concrete slightly decreased with the increase of the RCA replacement percentage. For the concrete with 100% RCA, a reduction of 6% is found for the cube compressive strength, while for the cylinder compressive strength, the reduction is 9%.

Table 6-2 – Properties of concrete mixtures

Mixture	$f_{cm,cub}$ [s] (MPa)	$f_{cm,cyl}$ [s] (MPa)	$f_{ctm,sp}$ [s] (MPa)	E_{cm} [s] (GPa)	Slump (mm)
RAC0	60.8 [0.5]	52.6 [0.8]	3.97 [0.23]	39.34 [0.92]	200
RAC10	60.1 [0.5]	50.8 [1.4]	4.12 [0.33]	38.90 [1.39]	200
RAC20	60.2 [0.7]	50.7 [1.2]	4.18 [0.16]	39.12 [0.82]	200
RAC50	58.7 [0.3]	49.2 [2.2]	3.91 [0.32]	37.91 [0.66]	190
RAC100	57.4 [0.6]	47.7 [1.9]	3.89 [0.31]	35.67 [0.56]	190

The decrease of the compressive strength for concrete (especially HSC) incorporating RCA is often considered to be caused by increased porosity of RCA and the effect of the additional interfacial transition zone (between the

original aggregate and attached mortar) in the concrete [15,154]. That negative effect becomes less significant with the improvement of the RCA quality [16]. The minor reduction in the compressive strength of the concrete in this study is mainly due to high-quality RCA. Table 6-2 also reveals that the ratio of the cylinder to cube compressive strength of the concrete mixtures varies from 0.83 to 0.87.

The use of RCA affects the tensile strength of concrete differently from that of the compressive strength. RCA generally has an angular shape; thus, a stronger bond between RCA and the (new) mortar in concrete can be developed than that between natural aggregate and mortar [77]. This can have a positive impact on the concrete tensile strength. From the test data in Table 6-2, it can be noticed that the influence of the RCA is not considerable. When the RCA replacement percentage is less than 20%, the concrete even exhibits higher tensile splitting strength than the reference concrete with no RCA.

6.3.3 Modulus of elasticity

It is widely reported that the use of RCA has a negative effect on the modulus of elasticity of concrete [83,85,90,91]. This is mainly because the modulus of elasticity of RCA is lower than that of natural aggregate. The measured data on the concrete modulus of elasticity, as shown in Table 6-2, also confirms earlier observations [83,85,90,91]. The modulus of elasticity of the concrete reduces as the RCA replacement percentage increases. A reduction of about 10% is noticed when the RCA replacement percentage is 100%.

6.3.4 Bond-slip curve

For each concrete mixture, a total of 6 pull-out specimens are tested. During the test, it can be noticed that with the increase of the pull-out load, the surface adhesion between the steel rebar and concrete is lost in a short time, and a slip at the free end of the rebar occurs, which represents the relative (local) slip between the steel rebar and concrete. With the increase of the pull-out load, the slip value increases gradually. The mechanical bearing of the steel rebar ribs against the concrete surface provides the primary source for the bond force to

resist the pull-out load. The friction between the concrete and rebar deformations (ribs) also plays a significant role in force transfer.



(a) RAC10



(b) RAC20



(c) RAC50

Figure 6-5 – Failure mode of the test specimen

No cracks in the concrete surface are observed at the maximum pull-out load, irrespective of the RCA replacement percentage. Upon that, the pull-out load decreases while the slip continues to increase. For each specimen, the test is stopped at a maximum slip of about 6 mm. As expected, all specimens exhibit

a pull-out failure of the steel rebars; that is, the steel rebars are slowly pulled out from the concrete while no splitting failure of the concrete is observed, regardless of the RCA content. After the tests, the specimens are cut into two pieces to examine internal failure. For all test specimens, it is found that the failure is a shear failure of the concrete around the ribs of the steel rebars. The incorporation of RCA in the concrete does not affect the failure mode of the specimen. Figure 6-5 shows the failure of some typical samples. The measured load-slip curves for each concrete mixture are shown in Figure 6-6. The following observations can be noticed from the figures:

- For each concrete mixture, some scatterings exist in the load-slip curves; however, the curves are consistent with each other;
- The maximum pull-out load of the test specimen incorporating RCA is similar to that of the reference specimen, indicating that the RCA does not considerably affect the maximum pull-out load;
- At the maximum load, the stress in the deformed steel rebar is smaller than the yield strength, which implies that no yielding of the rebar occurs during the test.

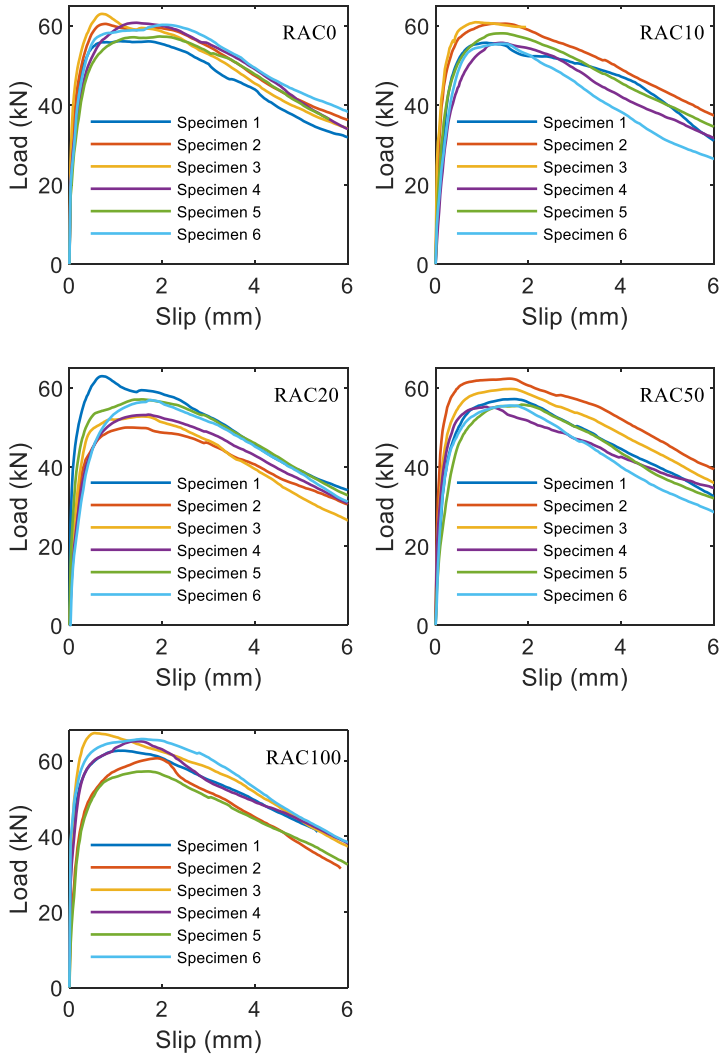


Figure 6-6 – Load-slip curves of the test specimen

Based on the average concept, the bond stress τ_b can be calculated by using Eq.(6-6):

$$\tau_b = \frac{F}{\pi \cdot d_b \cdot l} \tag{6-6}$$

where F is the pull-out load (kN); d_b is the diameter of the steel rebar (mm), and l is the embedded length ($l = 3d_b = 48$ mm).

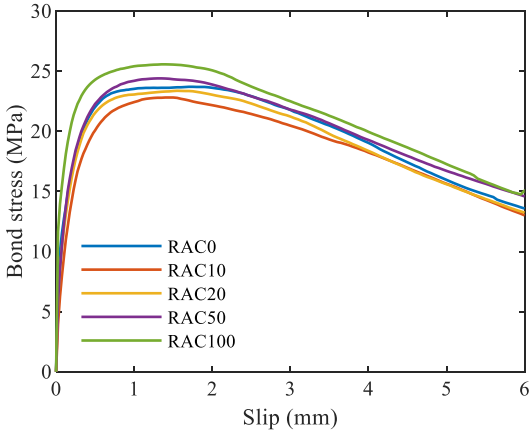


Figure 6-7 - A comparison of the mean bond-slip curve of the test specimen

Figure 6-7 compares the average bond-slip curve for each concrete mixture. The measured bond-slip curves exhibit a similar shape, irrespective of the replacement percentage of the RCA. This means the incorporation of the RCA does not considerably affect the general behaviour of the bond-slip curve. Based on the measured curve, the bond-slip process for each concrete mixture can be divided into three phases:

- Phase I: the relative slip is up to about 1.0 mm. In this phase, the surface adhesion between the rebar and concrete is firstly destroyed, and then the bond stress increases gradually with the increase of the relative slip;
- Phase II: the relative slip is between approximately 1.0 and 1.5 mm. In this phase, the slip increases dramatically, and the bond stress also

increases owing to the confining effect of the concrete cover. However, the increase of the bond stress is rather limited;

- Phase III: the relative slip exceeds around 1.5 mm. In this phase, the bond-slip curve enters a nonlinear descending branch due to reducing the bearing stress of the rebar ribs against the concrete surface.

6.3.5 Maximum bond stress

The measured bond strength, i.e., the maximum bond stress τ_{bmax} for each concrete mixture, is shown in Table 6-3. The test data reveals that the τ_{bmax} of the concrete is comparable, indicating that the RCA content does not have a considerable effect. The test findings are in good agreement with that reported in [106,132,141–143,133–140]. However, the test data implies that the standard deviation of the measured maximum bond strength is slightly higher when the RCA is used. Similar observations were also reported previously [106].

Table 6-3 - Bond strength of the concrete mixtures

Mixture	τ_{bmax} [s] (MPa)	s_{max} [s] (mm)	$\tau_{b,0.1}$ [s] (MPa)
RAC0	24.1 [1.8]	1.48 [0.5]	12.7 [2.9]
RAC10	23.3 [1.7]	1.33 [0.4]	9.5 [3.5]
RAC20	23.7 [2.2]	1.40 [0.4]	11.8 [3.8]
RAC50	24.6 [2.5]	1.47 [0.4]	12.3 [3.4]
RAC100	25.9 [2.1]	1.26 [0.5]	16.9 [3.5]

To better evaluate the effect of the RCA replacement percentage on the τ_{bmax}, it is necessary to filter out the effect of the concrete compressive strength since the RCA also affects the compressive strength of the concrete. For this purpose, a normalised bond strength $\tau_{bmax,n}$ was defined by Xiao and Falkner [136], which is calculated as the ratio of the τ_{bmax} to the square root of concrete compressive strength based on the equation for estimating the maximum bond stress in *fib* Model Code 2010 [82], as shown in Eq.(6-7).

$$\tau_{b\max,n} = \frac{\tau_{b\max}}{\sqrt{f_c}} \qquad (6\text{-}7)$$

The parameter $\tau_{bmax,n}$ enables us to compare the maximum bond stress of the RCA concrete and that of the reference concrete with the same compressive strength. The calculated normalised bond strength of the tested concrete mixtures in this study is shown in Figure 6-8. Note that the RCA replacement percentage in the figure refers to the substitution of the total coarse aggregate to achieve a consistent comparison with other results in the literature [106,136]. It is revealed that the normalised bond strength in general slightly increases with the increase of the RCA replacement percentage. It is also clear that the present results agree with previous findings [106,136]. This implies that RAC exhibits higher maximum bond stress than conventional concrete with natural aggregate when the compressive strength is similar.

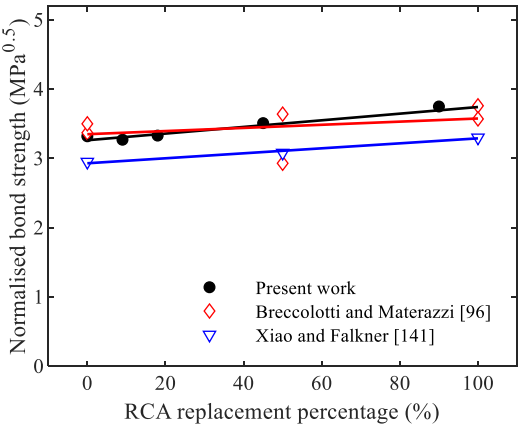


Figure 6-8 – Normalised bond strength of the concrete mixtures vs total coarse RCA replacement percentage

According to Xiao and Falkner [136], the enhanced bond strength for RAC is attributed to the fact that the modulus of elasticity of RCA is more close to that of cement paste. Prince and Singh [139] explained the positive effect of RCA on the $\tau_{bmax,n}$ in terms of the fracture toughness of concrete, as described by the concept of brittleness index (BI), which is computed as the ratio of the concrete compressive strength to its splitting tensile strength, i.e.,

$$BI = \frac{f_c}{f_{ct,sp}} \tag{6-8}$$

A higher brittleness index means a lower fracture toughness, indicating that the material is more brittle. Therefore, it can be expected that the bond strength of concrete should increase with a reduction of the brittleness index [139]. The calculated brittleness index values for the tested concrete mixtures in this study are presented in Figure 6-9. The test data reported in [32] are also shown in the figure. The brittleness index of the concrete seems to decrease slightly when the RCA replacement percentage increases. This helps explain why the normalised bond strength slightly increases with the increase of the RCA content.

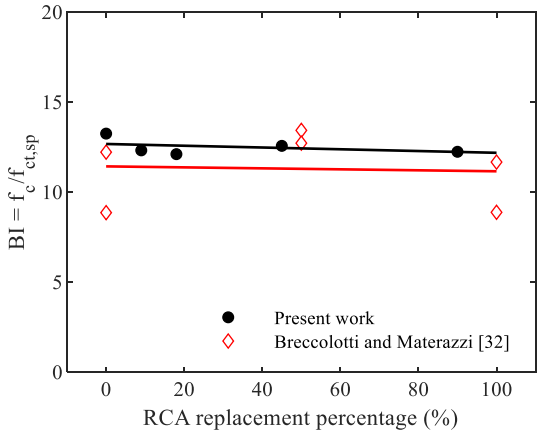


Figure 6-9 - Brittleness index of the concrete mixtures vs coarse RCA replacement percentage

Recently, Goudar et al. [155] reported that the steel-concrete interface properties, such as the porous zone thickness, affect the maximum bond stress. This can also explain the slight increase of 12% of the normalised bond strength when the RCA is fully incorporated. Since the density of RCA is smaller than that of normal coarse aggregate due to its porous surface caused by the attached mortar, the bleed water trapped between the rebar surface and concrete decreases, leading to reduced voids in the contact surface, which finally yields the increased maximum bond strength, especially for concrete with a large amount of the RCA. However, a more comprehensive study on the effect of

RCA on the contact surface between steel reinforcement and concrete is recommended in the future.

6.3.6 Slip at the maximum bond stress

It was reported by Siempu and Pancharthi [45] that the slip at the maximum bond stress s_{max} is affected by the concrete compressive strength, rebar diameter and rebar embedded length. The measured (free end) slip at the maximum bond stress s_{max} of the test specimens in this chapter are given in Table 6-3. It can be observed that neither the s_{max}-value nor its standard deviation of the RCA concrete specimens differs much from that of the reference concrete specimen. In Figure 6-10, the variation of the relative s_{max} (defined as the ratio of the s_{max} of the RCA concrete to that of the control concrete containing no RCA) with the replacement percentage of the RCA is presented. The data reported in [140,141] are also shown in the figure. It can be stated that, on average, the incorporation of RCA does not change the s_{max}.

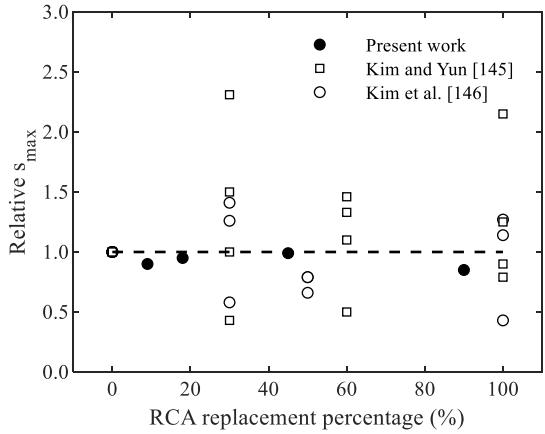


Figure 6-10 - Influence of RCA on the slip of the steel rebar

6.3.7 Bond stiffness

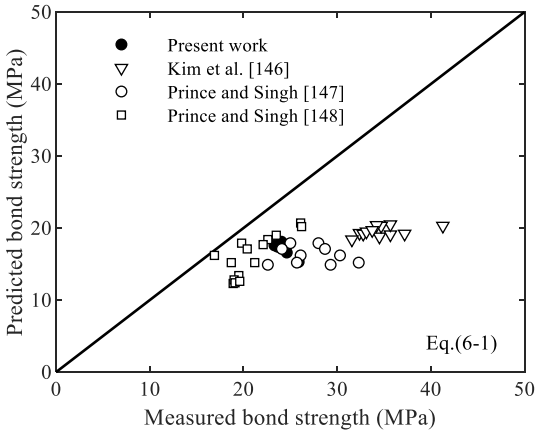
Following Metelli and Plizzari [156], the bond stiffness is evaluated in terms of the bond stress at a free end slip of 0.1 mm, denoted as $\tau_{0.1}$. The measured $\tau_{0.1}$ - values of the test specimens in the present study are presented in Table 6-3.

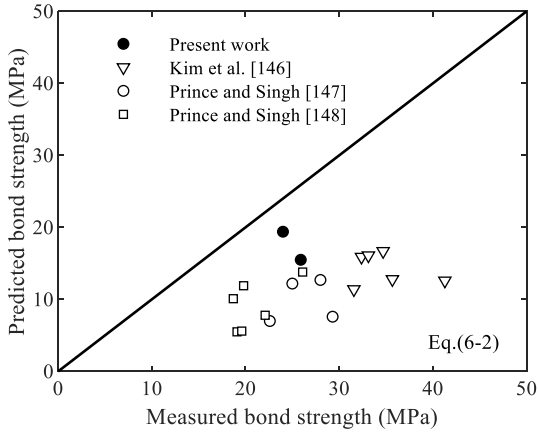
The RCA content does not have a remarkable effect on the bond stiffness and its standard deviation of the test specimens. Similar results were also reported by Prince and Singh [142].

6.4 Estimation of the maximum bond stress

6.4.1 Evaluation of existing models

Comparing the predictions of existing models for the maximum bond stress, as summarised before, with the measured data are shown in Figure 6-11 (a)-(e). This figure also includes previous test data on the bond between deformed steel rebars and HSC incorporating RCA reported in [141–143]. The figure indicates that the equation in *fib* Model Code 2010 [82] and those proposed by Siempu and Pancharthi [45], Seara-Paz et al. [47] and Kim et al. [144] underestimate the maximum bond stress while the model suggested by Kim et al. [144] tends to yield un-conservative predictions.




Present work
Kim et al. [146]
Prince and Singh [147]
Prince and Singh [148]
Eq.(6-2)
Predicted bond strength (MPa)
Measured bond strength (MPa)
0
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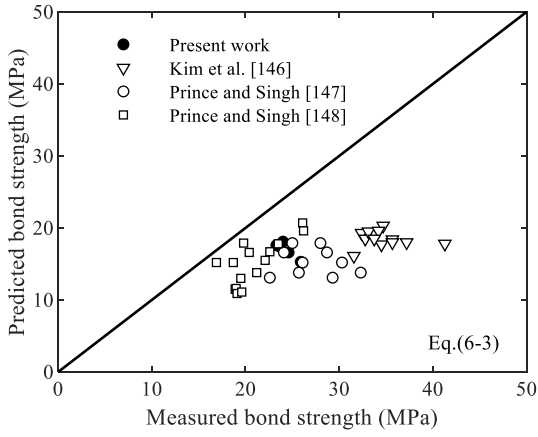
20
30
40
50


Present work
Kim et al. [146]
Prince and Singh [147]
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Eq.(6-3)
Predicted bond strength (MPa)
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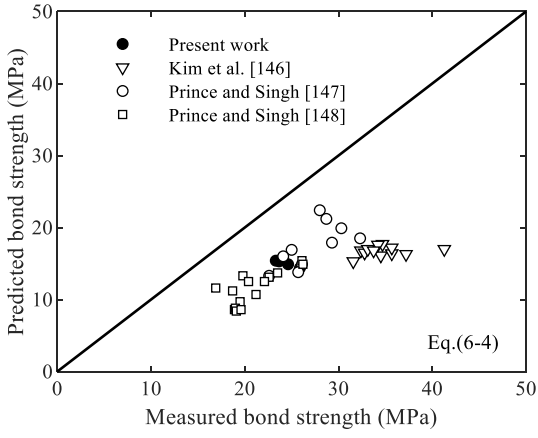
Present work
Kim et al. [146]
Prince and Singh [147]
Prince and Singh [148]
Eq.(6-4)
Predicted bond strength (MPa)
Measured bond strength (MPa)
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40
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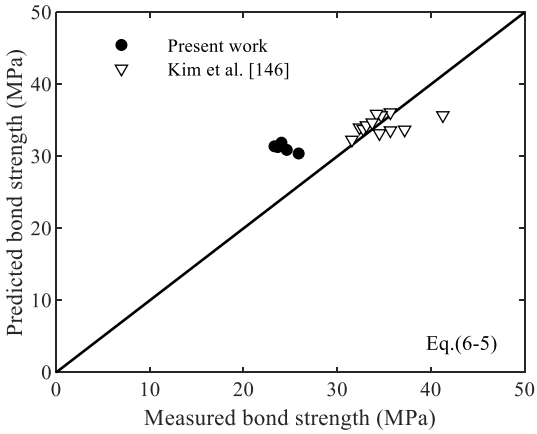

Figure 6-11 - Comparison between prediction models and experimental data

6.4.2 Proposal of a new model

The following equation is proposed to estimate the maximum bond stress between the steel rebars and concrete with the RCA based on the regression analysis of the present test data.

$$\tau_{b\,max}=3{,}26\sqrt{f_c}(1+0{,}165G) \qquad (6\text{-}9)$$

The proposed equation is a function of the concrete compressive strength and the replacement percentage G of coarse RCA (in volume). According to Eq.(6-9), the maximum bond stress of concrete with 100% RCA can be 14.5% higher than that of conventional concrete with natural aggregate, provided they have the same compressive strength. A comparison of the predictions of Eq.(6-9) with the present test data as well as that in [141–143] is shown in Figure 6-12. The proposed equation gives a good description of all the test data (correlation coefficient R = 0.85).

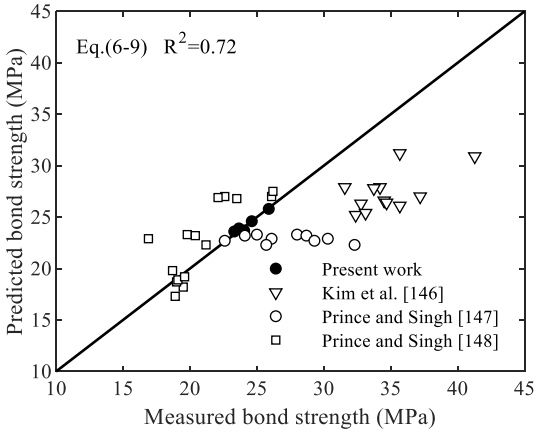

Figure 6-12 - Adaption of prediction models

6.5 Modelling of bond-slip relation

An analytical model for the bond-slip behaviour between steel rebar and concrete is required for numerical analysis and design of reinforced concrete components and structures. In the *fib* Model Code 2010 [82], the following model is recommended:

$$\tau_b = \begin{cases} \tau_{b\max}\left(\frac{s}{s_1}\right)^{\alpha} & (0 \le s \le s_1) \\ \tau_{b\max} & (s_1 \le s \le s_2) \\ \tau_{b\max} - \frac{(\tau_{b\max} - \tau_{bf})(s - s_2)}{s_3 - s_2} & (s_s \le s \le s_3) \\ \tau_{bf} & (s_3 < s) \end{cases} \qquad (6\text{-}10)$$

where s_1, s_2, s_3, α and τ_{bf} are parameters depending on the failure mode and bond conditions. In the case of the specimens tested in this chapter (pull-out failure and good bond conditions), s_1 = 1.0 mm; s_2 = 2.0 mm; s_3 is the clear distance between the ribs, which is 8.1 mm; α = 0.40 and τ_{bf} = 0.40 τ_{bmax}.

Figure 6-13 illustrates a comparison of the predicted bond-slip curve, according to Eq.(6-10), with the test data in this study. It should be noted that the measured maximum bond stress τ_{bmax} is used in the comparison. The model

given in *fib* Model Code 2010 [82] provides a good description of the test data, irrespectively the replacement percentage of the RCA.

However, it can be seen from the figure that the model in *fib* Model Code 2010 [82] yields a less satisfactory depiction of the bond-slip behaviour up to the slip s_1 (=1.0 mm), especially for the test specimen with high RCA content. This might relate to the performance of RAC subjected to multiaxial stress conditions. This can be investigated via three-dimensional nonlinear finite element simulation based on the actual triaxial properties of the concrete.

A more accurate description of the bond-slip behaviour up to the slip s1 can be achieved through adjusting the α-value. The predicted bond-slip behaviour using the optimised α-value for the specimens is also shown in Figure 6-13. In the figure, the used α-value is also presented. The prediction accuracy of the model is enhanced by selecting the appropriate α-value. However, the effect of the RCA replacement percentage does not show a clear trend. More research is required.

In previous studies, Xiao and Falkner [136] proposed a model to describe the bond-slip behaviour of RAC. Their model divides the bond-slip curve into the ascending branch (until the maximum bond stress) and the descending branch. The ascending branch is simulated using the proposal of Harajli [157], while the descending branch is described by the model of Guo [158] for describing the post-peak behaviour of concrete in uniaxial compression. In this chapter, the model by Xiao and Falkner [136] is also attempted to describe the present test data. In general, Xiao and Falkner's model reasonably describes the test data; however, some considerable discrepancies exist between the model predictions and test observations for the bond stress development in the vicinity of its maximum value. This behaviour can be observed in comparing the model simulations and test data in [136,142].

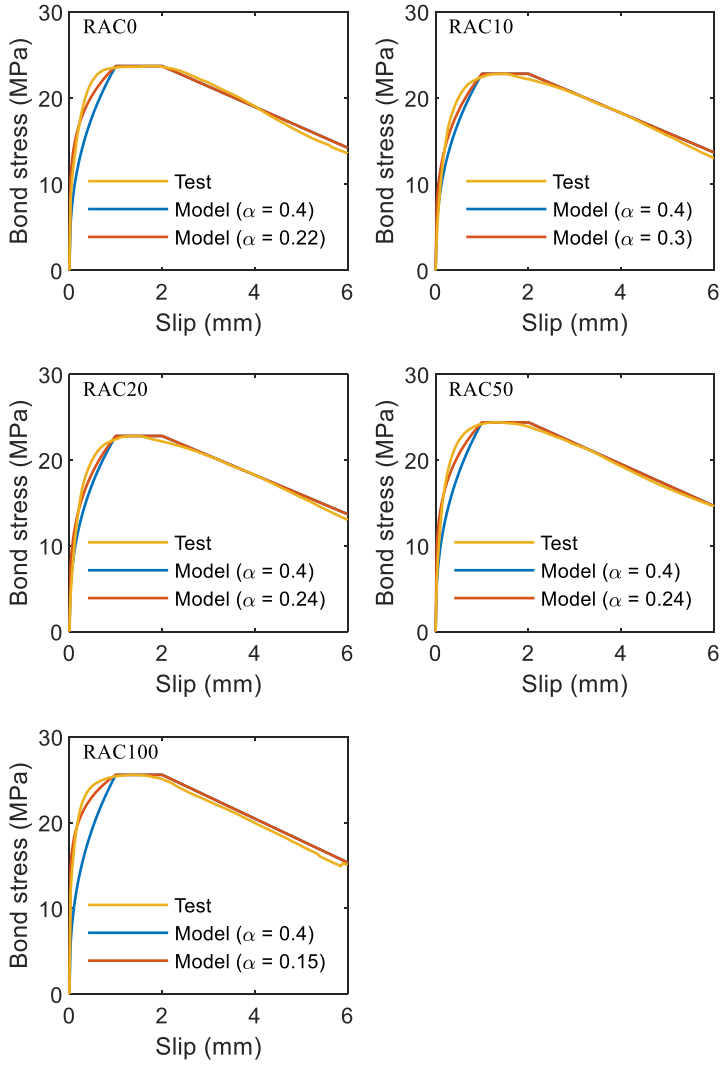

Figure 6-13 - Comparison between the measured bond-slip relation, fib Model Code 2010 bond-slip behaviour in Eq.(6-10) and the adapted bond-slip relation of the different mixtures

6.6 Conclusions

This chapter presents an experimental investigation on the bond of deformed steel rebars in concrete containing various amounts of industrially produced high-quality RCA, intending to examine the effect of the RCA on the bond characteristics and to provide a local bond-slip relation between deformed steel rebars and RAC that can be used in finite element simulations. Within the scope of this study, the following conclusions can be drawn.

The substitution of natural aggregate by the high-quality RCA negatively influences the concrete compressive strength and modulus of elasticity. With the increase of the RCA replacement percentage, the compressive strength and modulus of elasticity of the concrete decreases; however, the influence of the RCA on the concrete tensile splitting strength is not considerable.

The failure of the test specimens is characterised by the pull-out of the rebars from the test specimen, irrespective of the RCA content, and no splitting failure is observed. The results from the pull-out tests show that the RCA do not affect the maximum bond stress of the specimens; however, the normalised bond strength, defined as the ratio of the maximum bond stress to the square root of the concrete compressive strength, slightly increases with the increase of the RCA replacement percentage. The influence of the RCA on the slip at the maximum bond stress does not show a clear trend.

A new empirical model is proposed to estimate the maximum bond stress. The model is formulated as a function of the concrete compressive strength and RCA replacement percentage. The predictions of the proposed model are in good agreement with the test data in the present study as well as that in the literature.

The bond-slip model in fib Model Code 2010 [39] provides a satisfactory description of the local bond-slip behaviour of the test specimens in the present study, provided that the measured maximum bond stress is used. A more accurate description of the bond-slip behaviour up to the slip s1 (=1.0 mm) can be achieved by adjusting the model's α-value.

Chapter VII

Industrial tests

Up to this point, this PhD research has been focusing on laboratory tests on precast structural concrete with high-quality RCA. The knowledge obtained in the laboratory should be transferred into practice before the high-quality RCA can be implemented to produce precast structural concrete in practice. Different test series from Chapter 4 and Chapter 5 are repeated on an industrial scale to validate the obtained conclusions concerning the influence of high-quality RCA on the early-age mechanical properties and the variability of the compressive strength of precast structural concrete.

7.1 Test program

7.1.1 Mix design

The influence of RCA on the early age mechanical properties and the variability of the compressive strength of SCC is investigated on an industrial scale. The industrial SCC mixture with a design strength class of C50/60, described in 3.2.2, is used. In this mixture, the coarse fraction of aggregates is replaced by 20 and 100%$_{V/V}$ and compared to the reference concrete mixture's properties. The three mixtures are shown in Table 3-5. The concrete is made in a concrete precast plant according to the mixing procedure described in 3.3.2.

7.1.2 Test procedure

The lab tests described in Sections 4.2 and 5.2 are validated by tests on the same SCC precast mixture produced in a precast concrete plant. One pre-stressed beam, 44 cubes and 12 cylinders for each mixture are produced in one batch. The camber deflection of the pre-stressed concrete beam is analysed in Chapter 8. The associated test samples are used to determine the influence of

the RCA on the early-age mechanical properties and the variability of the compressive strength of the SCC mixture. The compressive strength is tested on cubes of 150x150x150mm^3 and cylinders of 150x300mm^3, according to EN 12390-3 [74]. The static elastic modulus is measured on cylinders of 150x300mm^3 according to B15-203 [76].

Twelve cubes and 12 cylinders per mixture are made and stored on top of the beam in the concrete plant, subjected to member curing conditions shown in Figure 7-1 and Figure 7-2. The 12 cubes are tested on different curing ages, 3, 7, 28 and 56 days, while three cylinders are tested at each curing age of 3,10, 28 and 59 days. All samples are transported and tested in the laboratory of KU Leuven Campus Bruges on the testing date.

The remaining 32 concrete cubes are carefully transferred after demoulding to the laboratory of KU Leuven Campus Bruges to cure in standard conditions in water (20°C). Three cubes are tested on each curing age of 3, 7, 14 and 56 days, respectively. Twenty cubes per mixture are tested at a curing age of 28 days to analyse the influence of RCA on the compressive strength variability.

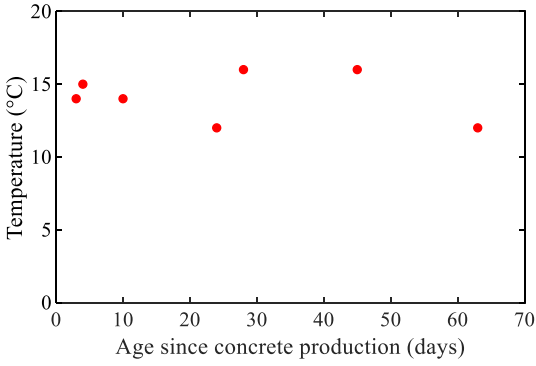

Figure 7-1 – Ambient temperature during storage of the concrete

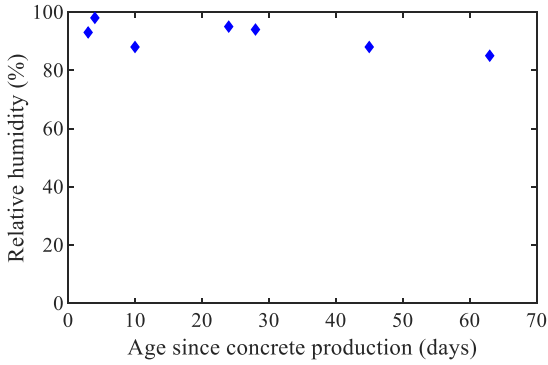

Figure 7-2 – Relative humidity during storage of the concrete beams

7.2 Early age mechanical properties of SCC with RCA

7.2.1 Workability

The workability of the mixtures is tested by measuring the slump flow value according to EN 12350-8 [88] and shown in Table 7-1. The measured slump flow values of the concrete mixtures are nearly equal for all mixtures due to the absorption water and superplasticizer of the type MasterEase 5035. No further tests on the workability and consistency of the self-compacting concrete are done.

Table 7-1 – Workability of RASCC

	RASCC0	RASCC20	RASCC100
Slump flow (mm)	720	720	720

7.2.2 Compressive strength

7.2.2.1 Water-cured samples

As mentioned before, 32 cubes are carefully taken to the lab in KU Leuven Campus Bruges after demoulding, 24h after concrete production. Three cubes are tested on each curing age of 3, 7, 14 and 56 days, while 20 cubes are tested

on 28 days. The mean test results of these standard-cured specimens are shown in Figure 7-3, and the mean values and standard deviation are mentioned in Table 7-2. The strength development of the three mixtures is quite similar. The mixture in which RCA entirely replace the coarse limestone fraction has an even higher strength on 7 and 14 days. The compressive strength on 28 and 56 days is similar for the three mixtures.

A comparison with the lab tests of the same mixtures, as discussed in Section 4.2, reveals a difference in strength between the lab and industrial tests. The industrial samples show an increase of 10% in compressive strength compared to samples made in lab conditions. The prescribed mix proportions and curing conditions should be equal. However, the weighing method is more accurate in lab conditions, as well as the determination of the water content of the used aggregates. Slight variations in the weighing method and water content during proportioning of ingredients explain the smaller slump flow (lab tests: 760mm – industrial tests 720mm) and result in higher compressive strength.

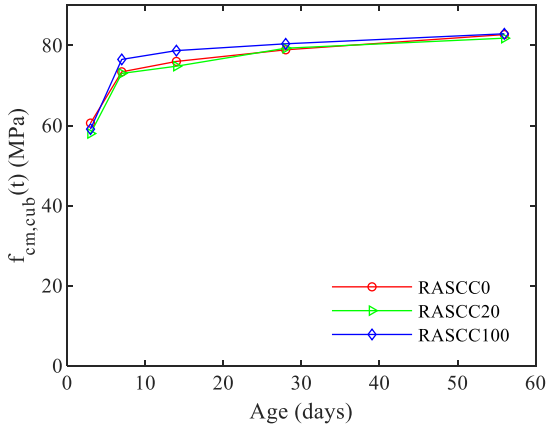

Figure 7-3 – Cube compressive strength development under standard-curing

Table 7-2 – Compressive strength development under standard-curing (MPa)

	$f_{cm,cub,3d}$ ($s_{fcm,cub,3d}$)	$f_{cm,cub,7d}$ ($s_{fcm,cub,7d}$)	$f_{cm,cub,14d}$ ($s_{fcm,cub,14d}$)	$f_{cm,cub,28d}$ ($s_{fcm,cub,28d}$)	$f_{cm,cub,56d}$ ($s_{fcm,cub,56d}$)
RASCC0	60.6 (1.3)	73.4 (2.2)	76.0 (0.8)	78.9 (2.2)	82.7 (1.2)
RASCC20	58.0 (0.5)	73.0 (2.7)	74.8 (3.1)	79.3 (2.2)	81.8 (4.5)
RASCC100	59.1 (1.2)	76.5 (3.4)	78.7 (2.5)	80.4 (3.3)	82.9 (3.1)

7.2.2.2 Member-cured samples

The precast beams in the concrete plant are demoulded after one day and stored in the production hall. It is practically impossible to store these large-scale beams in standard curing conditions. Therefore, several concrete cubes and cylinders were placed on the beams to recreate the same curing conditions. The temperature and relative humidity are measured on different moments, as shown in Figure 7-1 and Figure 7-2. Note that these values are not mean day-values but exact values at the moment of measuring.

Figure 7-4 and Figure 7-5 show the strength development of the concrete cubes and cylinders, respectively, subjected to member-curing. The difference between the three mixtures is negligible, considering the compressive strength variation of the samples. The mean values and standard deviation are shown in Table 7-3 and Table 7-4.

Table 7-3 – Cube compressive strength development under member-curing (MPa)

	$f_{cm,cub,3d}$ ($s_{fcm,cub,3d}$)	$f_{cm,cub,7d}$ ($s_{fcm,cub,7d}$)	$f_{cm,cub,28d}$ ($s_{fcm,cub,28d}$)	$f_{cm,cub,56d}$ ($s_{fcm,cub,56d}$)
RASCC0	62.1 (4.0)	73.1 (2.0)	80.4 (6.4)	83.7 (2.3)
RASCC20	58.0 (0.5)	72.0 (4.1)	79.3 (5.1)	83.7 (2.3)
RASCC100	59.1 (1.2)	73.0 (4.8)	80.7 (3.9)	84.1 (2.6)

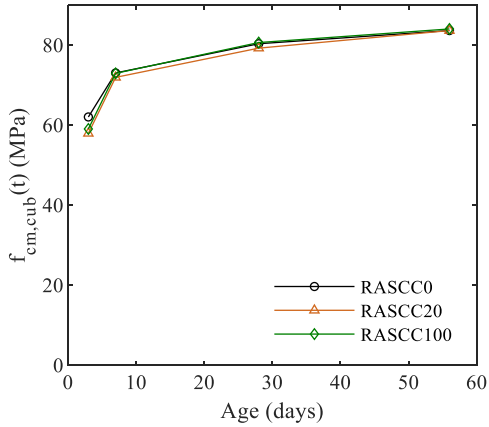

Figure 7-4 – Cube compressive strength development under member-curing

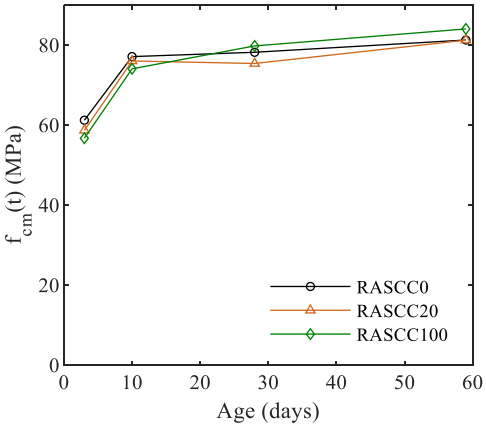

Figure 7-5 – Cylinder compressive strength development under member-curing

Table 7-4 - Cylinder compressive strength development under member-curing (MPa)

	$f_{cm,3d}$ ($s_{fcm,3d}$)	$f_{cm,10d}$ ($s_{fcm,10d}$)	$f_{cm,28d}$ ($s_{fcm,28d}$)	$f_{cm,59d}$ ($s_{fcm,59d}$)
RASCC0	61.3 (2.1)	77.2 (1.3)	78.3 (2.7)	81.3 (5.0)
RASCC20	58.8 (1.6)	76.1 (0.6)	75.5 (1.3)	81.3 (0.5)
RASCC100	56.8 (1.6)	74.1 (2.8)	79.9 (1.5)	84.1 (1.9)

7.2.2.3 Relative strength development

The relative strength development defined by the age factor of the three different mixtures is shown in Table 7-5 and Figure 7-6. A slight difference in the strength development between RASCC0 and RASCC100 is noticeable at three days for both member-cured and standard-cured samples. The presence of RCA slightly slows down the strength development of the concrete mixtures. This is similar to earlier lab tests concerning the early age mechanical properties of concrete with RCA in Chapter 4. The influence of RCA on the age factors is negligible at seven days and 28 days. Previous studies like [18,19,61,64,65] also indicate a slower strength development for up to 7 days. Comparing the strength development of member-cured and standard-cured samples does not show a considerable difference.

In EN 1992-1-1 [1], the compressive strength of concrete at any time $f_c(t)$ is calculated as in Eq.(4-1). In ACI 209 [2], Eq.(4-2) is recommended to estimate the compressive strength of concrete at any time $f_c(t)$. A comparison of Eq.(4-1) and Eq.(4-2) with the test data of standard-cured and member-cured samples is also given in Figure 7-6. Both models underestimate the development of the RAC compressive strength in time. However, the equation in EN 1992-1-1 yields slightly more accurate results than the ACI 209 equation.

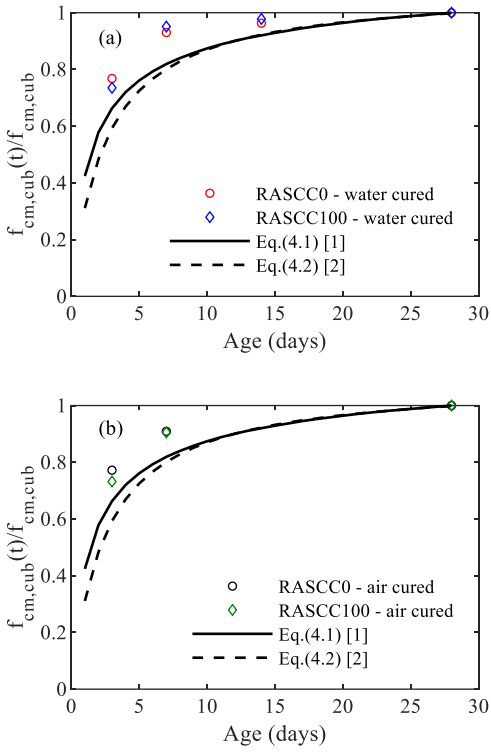

Figure 7-6 - Age factor for compressive strength of concrete in time

Table 7-5 – Age factor of compressive strength of concrete in time

	Member-cured			Standard-cured		
	$f_{cm,cub,3d}/f_{cm,cub}$	$f_{cm,cub,7d}/f_{cm,cub}$	$f_{cm,cub,28d}/f_{cm,cub}$	$f_{cm,cub,3d}/f_{cm,cub}$	$f_{cm,cub,7d}/f_{cm,cub}$	$f_{cm,cub,28d}/f_{cm,cub}$
RASCC0	0.77	0.91	1	0.77	0.93	1
RASCC20	0.73	0.91	1	0.73	0.92	1
RASCC100	0.73	0.91	1	0.74	0.95	1

7.2.2.4 Industrial validation of lab results

Figure 7-7 shows a comparison between the age factor of the compressive strength for SCC with RCA, made in the lab and on an industrial scale. The results from the lab tests have been discussed in Section 4.2. The figure shows a general tendency of a lower age factor for mixtures with RCA on a curing age of three days, irrespective of the curing conditions. The difference in strength development is negligible at seven days.

The test data is compared to the strength prediction models from Eurocode 2 – [1] and ACI 209 [2], showed in Eq.(7-1) and Eq.(7-2), respectively.

$$f_c(t) = \exp\left\{s\left[1-\left(\frac{28}{t}\right)^{\frac{1}{2}}\right]\right\} f_{cm} \qquad (7\text{-}1)$$

in which

s = 0.20 for rapid hardening cement: CEM 42.5R, CEM 52.5N and CEM 52.5R

$$f_c(t) = \frac{t}{a+b\cdot t} f_{cm} \qquad (7\text{-}2)$$

in which a = 2.30 and b = 0.92 for concrete with rapid hardening cement and moisture-cured concrete specimens

Figure 7-7 shows that both prediction equations underestimate the compressive strength development. The constants in both equations are set up to estimate the strength development of rapid hardening cement types. However, these constants still cover several rapid hardening cement types and therefore give a conservative prediction of the strength development when CEM I 52.5 R is used in precast concrete.

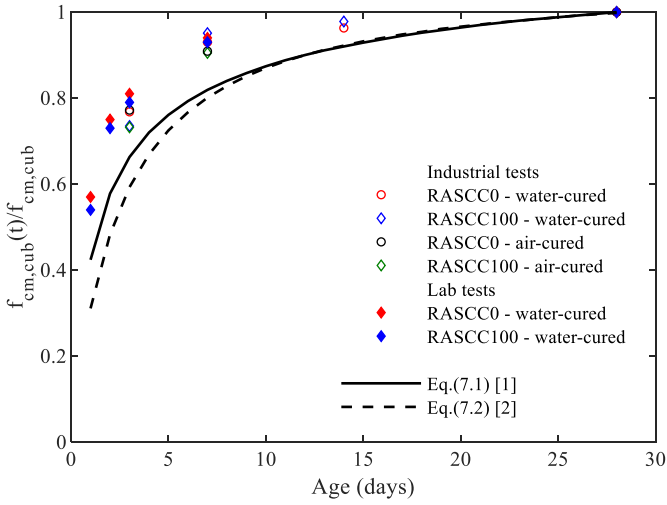

Figure 7-7 – Comparison of age factor for compressive strength of samples in industrial tests and lab tests with prediction equations

A more accurate description of the strength development of the used precast concrete types can be achieved by adjusting the constants in Eq.(7-1) and Eq.(7-2). Via the least-square method, adapted constants are proposed, which provide a better estimation of this type of concrete's strength development. Table 7-6 gives the adapted constants, while Figure 7-8 shows a fair comparison of the test data with Eq.(7-1) and Eq.(7-2) with adapted constants. The references, e.g. Brandes and Kurama [38] and Gonzalez and Etxeberria [46], also use CEM I 52.5 R in their mixtures. Their test data is also included in Figure 7-8. The equations Eq.(7-1) and Eq.(7-2) with the adapted constants seem to accurately predict precast concrete strength development using high-early strength cement.

Table 7-6 – Proposed constants in Eq.(7-1) and Eq.(7-2)

	Current constants	Proposed constants
Eurocode 2 [1]		
s	0.20	0.12
ACI 209 [2]		
a	2.30	0.84
b	0.92	0.96

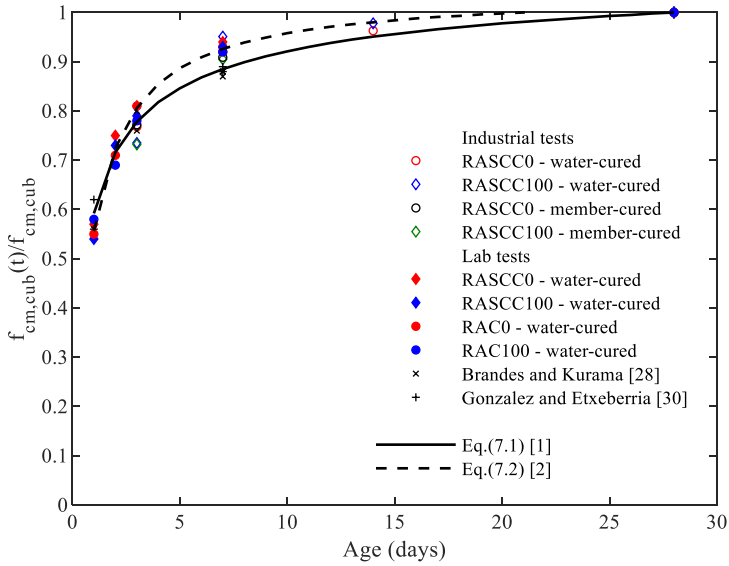

Figure 7-8 - Comparison of age factor for compressive strength of samples in industrial tests and lab tests with prediction equations with adapted constants

7.2.3 Static elastic modulus

7.2.3.1 Industrial tests

The tests on the modulus of elasticity of concrete (E_c) in this section are performed according to B15-203 [76]. The E_c of the mixtures is presented in Figure 7-9. Three member-cured cylinders are tested at each curing age, which

is 3, 10, 28 and 59 days. The figure shows that the measured E_c value of each concrete mixture increases with the curing time. The increase mainly occurs in the first ten days. A replacement of $20\%_{V/V}$ of the coarse limestone fraction does not influence the development of the elastic modulus. From the results of RASCC100 can be concluded that the RCA has nearly no effect on the static elastic modulus on 28 and 56 days. At 3 and 10 days, it seems that the E_c of RASCC100 is 4% lower than that of the other mixtures. This is similar to the compressive strength results, where the presence of RCA results in slower strength development. From the results shown in Figure 7-10, the age factor of RASCC100 is lower in the first ten days, indicating a slower development of the static elastic modulus.

EN 1992-1-1 [1] recommends Eq.(4-8) to estimate the $E_c(t)$ of concrete based on E_{cm}. In ACI 209 [2], Eq.(4-9) is used to describe the relation between $E_c(t)$ and E_{cm} for concrete. The predictions of Eq.(4-8) and Eq.(4-9) are also shown and compared to the experimental data in Figure 7-10. Eq.(4-8) provides a relatively accurate description of the concrete's E_c development with time, while Eq.(4-9) is more conservative in predicting the increase of the E_c in time, especially before seven days.

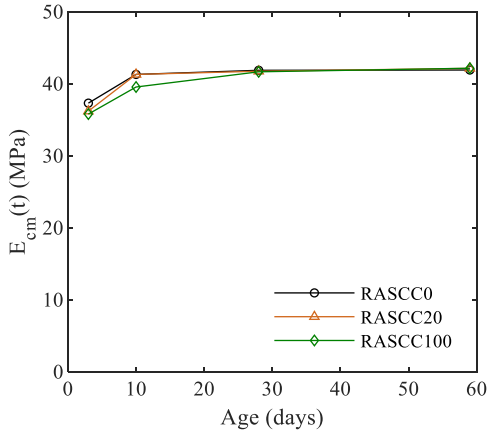

Figure 7-9 – Static elastic modulus development member curing

Table 7-7 – Static elastic modulus development member curing (MPa)

	$E_{cm,3d}$ ($s_{Ecm,3d}$)	$E_{cm,10d}$ ($s_{Ecm,10d}$)	$E_{cm,28d}$ ($s_{Ecm,28d}$)	$E_{cm,59d}$ ($s_{Ecm,59d}$)
RASCC0	37.3 (0.5)	41.3 (0.5)	41.9 (0.6)	41.9 (0.9)
RASCC20	36.2 (1.0)	41.3 (0.8)	41.7 (0.9)	42.1 (1.3)
RASCC100	35.8 (1.2)	39.6 (1.1)	41.7 (0.6)	42.1 (1.8)

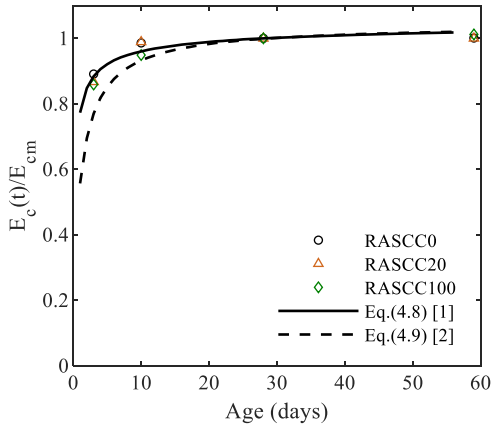

Figure 7-10 - Age factor elastic modulus

Figure 7-11 presents the relation between E_c and $f_{c,cyl}$, applied on the test data on 28 days. The experimental data is compared to prediction equations Eq.(4-10) from ACI 209 [2], Eq.(4-11) from Eurocode 2 [1] and Eq.(4-12) from JCI [87]. As illustrated in Figure 7-11, the equation by Eurocode 2 slightly underestimates the E_c calculated by $f_{c,cyl}$, while the equation by JCI overestimates E_c. The model given by ACI 209 has the best fit with the measured data.

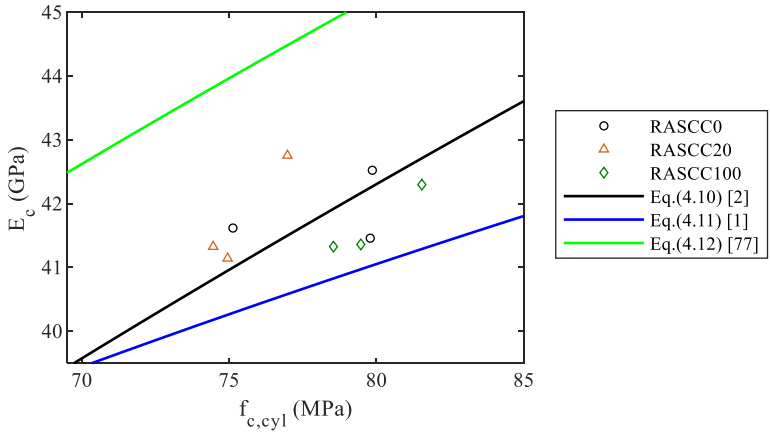

Figure 7-11 - Relation between elastic modulus and compressive strength on 28 days

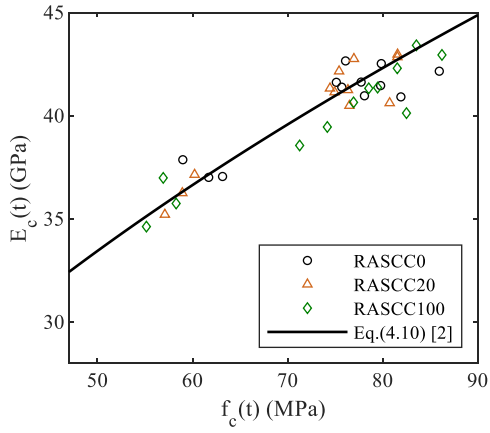

Figure 7-12 - Relation between elastic modulus and compressive strength

In ACI 209 [2], Eq.(4-10) is proposed to estimate the E_c of concrete at any time based on the concrete compressive strength at the same age. This equation is compared with the dataset in this thesis in Figure 7-12. A good relationship between the data and Eq.(4-10) can be concluded for all the curing ages.

7.2.3.2 Industrial validation of lab results

Figure 7-13 shows a comparison between the age factor of the compressive strength for SCC with RCA, made in the lab and on an industrial scale. The results from the lab tests have been discussed in Section 4.2. The figure shows a similar tendency as the compressive strength results, which is a lower age factor of the elastic modulus for mixtures with RCA on a curing age of 3 days for both lab and industrial tests. The difference in strength development is negligible at seven days for the lab tests, while incorporating RCA in SCC made in the industrial precast plant still leads to a lower age factor of the elastic modulus at a curing age of 10 days. Eq.(4-8) accurately describes concrete development with and without RCA, but Eq.(4-9) provides more conservative predictions.

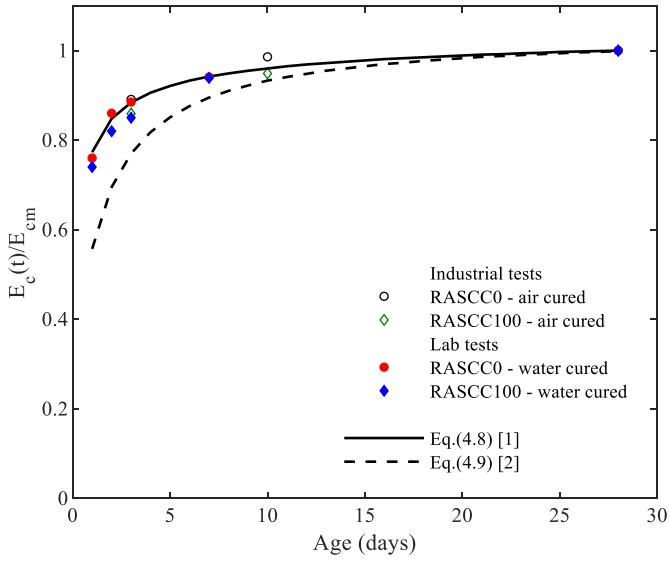

Figure 7-13– Comparison of age factor for elastic modulus in industrial tests and lab tests

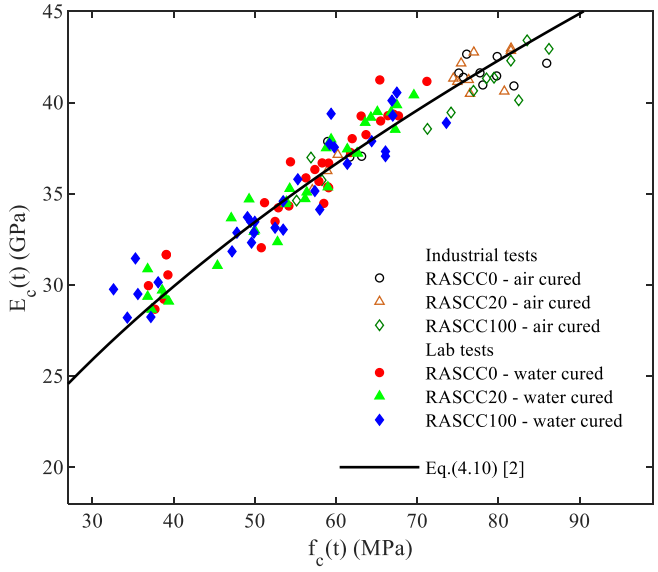

Figure 7-14 – Relation between elastic modulus and compressive strength for industrial and lab tests

As mentioned before, Eq.(4-10) was proposed in ACI 209 [2] to estimate the E_c of concrete at any time based on the concrete compressive strength at the same age for normal-weight concrete. This equation is compared with all data from lab and industrial tests on SCC with RCA in Figure 7-14. A good relationship between the data and Eq.(4-10) can be observed for all curing ages. The results from the lab tests and industrial tests both follow the same relation between compressive strength and elastic modulus.

Previous research has shown that the relation between E_c and f_c changes with RCA incorporation in the concrete mixtures [159,160]. In general, it is expected that for a similar f_c, the E_c is lower for mixes with a higher replacement percentage of RCA [161]. *Fib* Model Code [82] defines the relationship between elastic modulus and compressive strength of concrete by Eq.(4-13):

$$E_c = 21.5\ \alpha_E\ (f_c/10)^{0.33} \qquad (7\text{-}3)$$

In Eq.(4-13), α_E is defined as a factor incorporating the influence of the different types of aggregates on the relation between compressive strength and elastic modulus. Section 4.1.3.4 already showed that the relation between Ec and fc changes in terms of α_E value for the conventional precast mixture. The change of α_E value of the SCC precast mixtures is shown in Table 7-8. The change is minimal for both lab and industrial tests. The SCC precast mixture uses less RCA when considering the whole aggregate skeleton (see Section 3.2.2), explaining the slight differences between the different mixtures.

Table 7-8 - α_E for different mixtures

Mixtures	α_E
RASCC0 – Lab tests	1.00
RASCC20 – Lab tests	0.99
RASCC100 – Lab tests	0.99
RASCC0 – Industrial tests	1.00
RASCC20 – Industrial tests	1.00
RASCC100 – Industrial tests	0.99

7.3 Variability of f_c of SCC with RCA

A total of 20 cubes per mixture are tested on the 28 days compressive strength. The statistical parameters of the test data, including the median compressive strength $\widetilde{f_c}$, the mean compressive strength $f_{cm,cub}$, the standard deviation s and the coefficient of variation CoV of the compressive strength of all the tested specimens are summarised in Table 7-9.

Table 7-9 – RASCC – industrial tests – statistical parameters

	$f_{c,max}$ (MPa)	$f_{c,min}$ (MPa)	$\widetilde{f_c}$ (MPa)	$f_{cm,cub}$ (MPa)	s (MPa)	CoV (%)	$f_{cm}/f_{cm,RAC0}$
RASCC0	84.0	75.7	79.0	78.9	2.2	2.8%	1.00
RASCC20	81.8	72.9	79.9	79.3	2.2	2.8%	1.01
RASCC100	85.3	74.0	81.2	80.4	3.3	4.1%	1.02

Considering the standard deviation, the mixtures made in the precast concrete plant appear to have a similar compressive strength, as shown in Table 7-9. The standard deviation and CoV stay constant for a replacement percentage of RCA up to 20%$_{V/V}$, while an increase of these parameters is noticed as the replacement percentage increases to 100%$_{V/V}$. These results are similar to previous lab tests concerning the within-batch variability of RASCC mixtures. From lab results discussed in Section 5.1 and industrial test results above is observed that the incorporation of RCA in SCC only has a minor influence on the mean compressive strength, while the variability of the compressive strength seems to increase when RCA replaces 100%$_{V/V}$ of the coarse fraction of limestone. However, the differences between the mixtures are relatively small. A statistical analysis is carried out to distinguish statistically significant differences.

7.3.1 Statistical analysis

7.3.1.1 Chi-Square test

The Chi-Square test is used to determine whether the compressive strength results on 20 cubes per mixture can fit a normal distribution [119,120]. More information about this test can be found in Section 5.2.1. The dataset's ability to fit the normal distribution is necessary for future statistical tests. The null hypothesis states that the data follows a normal distribution. The null hypothesis can be rejected if the χ^2-statistic per data sequence exceeds the χ^2-critical. Table 7-10 shows the three series of data results, where it can be accepted that all test data are normally distributed.

Table 7-10 – Chi-square test results for normal distribution

Mixture	χ^2-statistic (calculated)	χ^2-critical (at 0.05 level)	P-value	Accept or reject H_0
RASCC0	6.05	11.07	0.30	Accept
RASCC20	9.16	11.07	0.10	Accept
RASCC100	10.85	12.59	0.15	Accept

7.3.1.2 Probability density functions

The histograms and probability density function of the different datasets are shown in Figure 7-15. The probability density function widens when 100%$_{V/V}$ replacement percentages are used. This is the result of a higher standard deviation. The confirmed similarity from the dataset with a normal distribution can be seen in the histograms' resemblance with the probability density function.

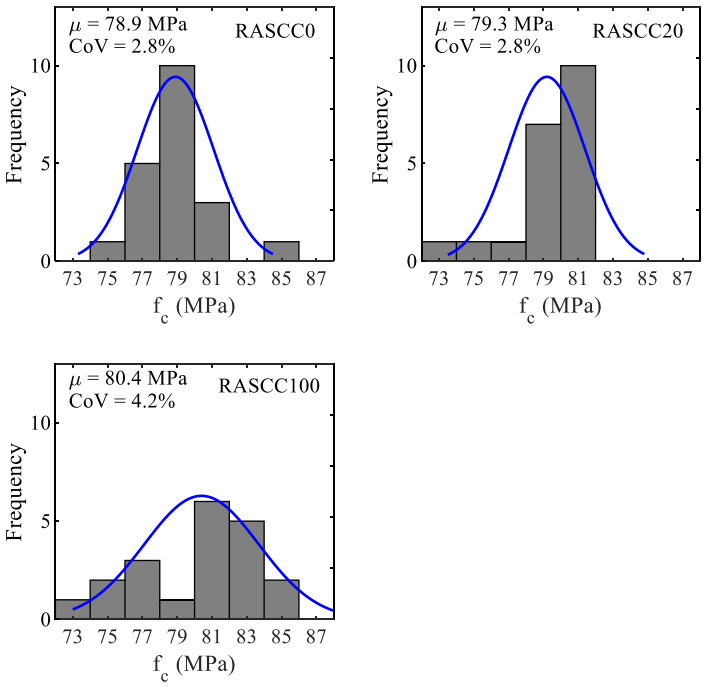

Figure 7-15 – Histogram and distribution density function for the compressive strength of SCC with RCA

7.3.1.3 F-test

The F-test is used to check statistically significant differences between the variation of the different mixtures [119,120]. The null hypothesis is defined in Eq.(5-6) in Section 5.2.3. The variation between the two mixtures is significantly

different if the null hypothesis is rejected. More information on the F-test can be found in Section 5.2.3. Table 7-11 shows the F-test results on the SCC with RCA. The larger standard deviation of RASCC100 compared to the other can be considered statistically significantly larger. The variation on the test data of RASCC0 and RASCC20 can be considered similar. This is similar to the F-test results on the mixtures made in the lab. The industrial tests can confirm the conclusion made in Chapter 5 that the RASCC mixtures can have a replacement percentage of up to 20% without significantly increasing the compressive strength variability. This is similar to test data from [104], [105] and [109].

Table 7-11 - F-test results of compressive strength on industrial SCC with RCA

Comparison between different variances	F-value (calculated)	F-critical (at 0.05 level)	Accept or reject H_0
s_{RASCC0}-$s_{RASCC20}$	0.98	2.12	Accept
s_{RASCC0}-$s_{RASCC100}$	2.28	2.12	Reject

7.3.1.4 ANOVA & Tukey test

The ANOVA and Tukey test distinguish significant differences between the tested mixtures' mean values, as explained in Section 5.2.4 [119,120]. The ANOVA test revealed statistical differences between the different mixtures ($F(2,57) = 3.16$, $p=0.19$). More details about the ANOVA tests are found in Table 7-12.

Table 7-12 - One way ANOVA: 28 days compressive strength of SCC with RCA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	24.18	2	12.09	1.73	0.19	3.16
Within Groups	398.90	57	7.00			
Total	423.08	59				

Table 5-18 shows the results of the Tukey test. The null hypothesis is rejected when the difference in the mean value of the compressive strength of SCC between the mixture is larger than the calculated T-value. The Tukey tests

show that the differences in the mean value of the compressive strength between the mixtures are statistically insignificant. Incorporating RCA in this SCC mixture up to 27% of the total aggregate skeleton, 90% of the coarse fraction, and 100% of the coarse limestone fraction does not affect the compressive strength's mean value. These findings are similar to the ANOVA and Tukey tests on mixtures made in the lab, described in Chapter 5.

Table 7-13 - Tukey test results on 28 days compressive strength of SCC with RCA

Comparison between mixtures	T-value	Difference between mean	Accept or reject H_0
RASCC0 – RASCC20	2.31	0.40	Accept
RASCC0 – RASCC100	2.31	1.50	Accept

7.4 Conclusions

This chapter presents the analysis of the test results of an industrial test campaign testing the influence of RCA on the early-age mechanical properties and the variability of the compressive strength of SCC with RCA. The tested industrial mixtures are similar to the mixtures made in the lab to validate earlier analysis on the influence of RCA on SCC properties.

The high-quality RCA slightly influence the strength development of SCC. This confirms the results of the lab tests, where the effect is obvious before a curing age of 3 days. Both lab-made SCC and SCC made in the precast plant show that the strength development of SCC with RCA is slightly slower compared to SCC with NA. Despite the slight difference in strength development, strength development models from Eurocode 2 [1] and ACI 209 [2] can still be used if RCA is used in concrete, but they turn out to be somewhat conservative for the prediction of the compressive strength of precast concrete using CEM I 52.5 R. New coefficients for Eq.(7-1) [1] and Eq.(7-2) [2] are proposed and validated by own data and data from other studies. The development of the elastic modulus is similar to the compressive strength development. Minor differences are noticed at a curing age of three days. ACI 209 [2] is somewhat conservative towards the development of the elastic modulus, while Eurocode 2 accurately

describes its development of the static elastic modulus. This accounts for both lab results as industrial test results.

The relation between the elastic modulus and the compressive strength proposed by ACI 209 [2] creates the best fit with the test data. The incorporation of RCA does not influence this relationship due to the high quality of RCA and limited replacement percentage to the complete aggregate skeleton. This statement is valid for a testing age of 2 to 28 days.

The test results show no statistically significant influence of RCA on the mean value of the compressive strength of SCC after 28 days subjected to water curing. Even a complete incorporation of RCA in SCC, made in a precast concrete plant, does not affect the compressive strength's mean value. The compressive tests on 20 cubes at a curing age of 28 days showed a decrease of 2% in compressive strength when the coarse fraction of limestone is entirely replaced by RCA. The compressive tests on cylinders showed no difference between the different mixtures. Based on the lab and industrial tests results, it can be concluded that the incorporation of high-quality RCA has nearly no influence on the compressive strength of SCC. The RCA's negative influence is minimised by the high quality of RCA and the smaller replacement percentage of the coarse limestone fraction relative to the complete aggregate skeleton, which is only $27\%_{V/V}$.

The within-batch variability of the compressive strength of SCC with RCA is determined using the standard deviation and coefficient of variation. For both lab tests and industrial tests, it is observed that incorporating $20\%_{V/V}$ RCA does not influence the variability of the test results. A replacement percentage of $100\%_{V/V}$ results in a statistically significant higher variability. This conclusion confirms the lab test results with the precast SCC mixture.

Chapter VIII

Camber of prestressed concrete with RCA

This chapter assesses the feasibility of using high-quality RCA in precast prestressed concrete by producing full scale pretensioned prestressed concrete beams with different percentages of high-quality RCA. The upward deflection of the manufactured beams caused by releasing the prestress is measured and analysed. Different prediction models are compared with the test data.

8.1 Introduction

The use of precast prestressed concrete beams in the construction industry is increasing nowadays. This production system includes several significant advantages. Prestressed concrete reduces tensional forces in the service state by applying a compressive force in the longitudinal direction of the beam before applying external loads. The most considerable advantage of prestressed concrete is the reduction of the beams' dimensions, reducing the used materials for similar or larger span lengths [162]. Also, the concrete quality can be monitored and kept more stable since this type of concrete is produced in a controlled environment. This leads to better quality control and larger standardised products.

In Flanders, Belgium, the most commonly used type of prestressed concrete is pretensioned concrete. In this case, the tendons are firstly tensioned before the concrete is poured into the moulds. Most precast plants in Flanders use the

Hoyer method in the production process of prestressed beams. Several beams are produced in line on the same prestress bed, using the same prestressed strands. This method allows precast companies to improve their production efficiency [162].

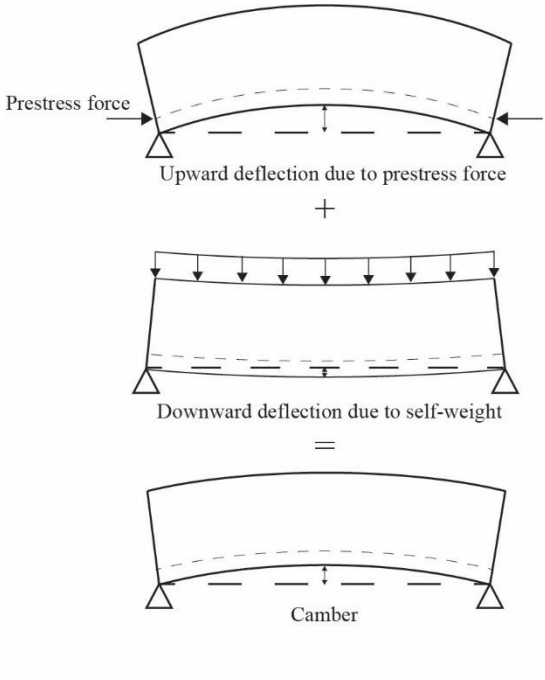

Figure 8-1 – Camber of prestressed concrete beams

After the concrete pouring, the beams are cured for 2 to 3 days before applying the prestress force onto the concrete beams. After the prestress transfer, the prestressing force causes an upward deflection. This deflection depends on the position of the prestressing strands, the concrete properties and the initial prestressing force. The beams' self-weight will cause a downward deflection, depending on the dimensions of the beam and the material properties [162]. The resultant deflection is calculated by subtracting the deflection due to the beams' self-weight from the deflection due to the prestressing force. The terminology of these types of deflections can differ among different studies [162,163,172–178,164–171]. In this work, the resultant deflection is called 'camber', which is

the combination of the upward and downward component, as shown in Figure 8-1 [163]. This terminology has been adopted in most recent studies [167,169-172,174,176-178,180,181].

The initial camber is measured directly after the prestress force is transferred on the beam. This value serves as a basis for the prediction for the long-term camber. The upward and downward components of the camber change in time and are affected by many factors. The total camber $\Delta_c(t)$ represents the sum of the initial camber Δ_i and the long-term camber $\Delta_{lt}(t)$ as shown in Eq.(8-1) and Figure 8-2. The long-term camber depends on the creep and shrinkage of concrete, relaxation of the prestressing strands and loss of the prestress force.

$$\Delta_c(t) = \Delta_i + \Delta_{lt}(t) \tag{8-1}$$

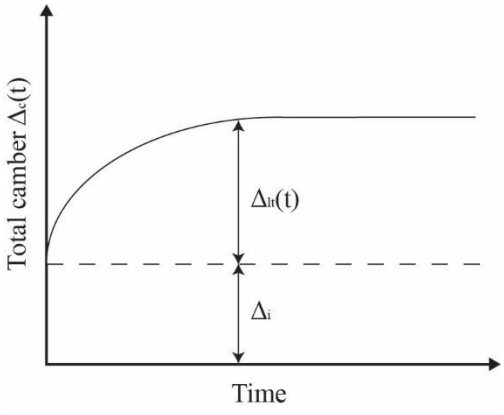

Figure 8-2 – Total camber of prestressed beams

An accurate prediction of the camber in precast pretensioned concrete beams has been an important issue that precast companies and contractors have to consider. An overestimation or underestimation of the camber can result in difficulties in composing the structure [177]. As a result, inaccuracies in camber predictions can cause problems in the service state of structures. The problems mainly come from having discrepancies of camber on adjacent beams. In most cases, the difference between the design camber and the measured value can be

significant. Inaccuracies in the prediction of the initial camber can affect the prediction of the long-term camber since this is, in most cases, based on the initial camber [162].

As stated before, the precast concrete industry, just like other sectors, is eager to move towards more sustainable material use. Recycling high-grade concrete waste into high-quality RCA for precast concrete elements can help move towards more sustainable precast products. RCA in concrete has been a widely used research topic in the last decade [43,179]. Its use in non-structural applications has been proven, while more research is needed to implement RCA in structural applications, as already discussed in Section 5.3.4. Some studies discuss RCA in conventional structural concrete and have shown that incorporating RCA in concrete can lead to higher creep and shrinkage deflection than conventional concrete with natural aggregates. Furthermore, several load tests on reinforced concrete beams incorporating RCA showed that immediate and long-term service load deflections increase with an increased RCA replacement percentage [180–182]. In this regard, it is important to map the effect of RCA on the camber of prestressed concrete beams to assess the feasibility of using RCA in precast prestressed concrete elements.

8.2 State of the art

Despite an upcoming demand for RCA in concrete structural applications, using high-quality RCA in prestressed structural concrete has rarely been dealt with. In 2013, Breccolotti and Materazzi [183] briefly evaluated the use of RCA in prestressed concrete by doing a reliability analysis based on the different properties of RCA compared to natural aggregates. The authors concluded that the use of low incorporation percentages (<20%) in prestressed concrete is acceptable without compromising the performance of the prestressed elements. Higher replacement percentages (>50%) require a higher prestress force to counter the negative effect of RCA on concrete properties. In 2016, Gonzalez-Corominas et al. [32] performed a comparative evaluation of the structural behaviour of prestressed concrete railway sleepers using high-quality RCA as a replacement of natural aggregates. The authors concluded a satisfactory

performance for all sleepers on a static load test, even when RCA completely replaced the coarse natural aggregates. Brandes and Kurama published several articles on the results of a large research project on the behaviour of prestressed concrete with RCA [38,39,184]. The test results indicated that the incorporation of high-quality RCA does not influence the service load behaviour of the beams and the behaviour under ultimate loads. The authors concluded that current standard design methods to predict the deflections of prestressed concrete beams under different conditions were not affected by the incorporation of high-quality RCA.

Although the promising research results above, there are still many issues about using high-quality RCA in prestressed concrete. The influence of high-quality RCA on different aspects of structural concrete on the material level has been evaluated on lab scale in previous chapters. To further evaluate the feasibility of using high-quality RCA in precast prestressed structural concrete elements, it is necessary to transfer the laboratory knowledge into practical situations. To the best of the author's knowledge, no information on the influence of high-quality RCA on the camber of prestressed concrete beams is available in the literature.

8.3 Test program

8.3.1 Mix design

The industrial SCC mixture with a design strength class of C50/60, described in Section 3.2.2, is used as a basis to produce the prestressed concrete beams. Three different mixtures, RASCC0, RASCC20 and RASCC100, described in Table 3.5, are used to produce the prestressed concrete beams and to investigate the influence of high-quality RCA on the camber of the beams. The mixing procedure is specified in Section 3.2.

8.3.2 Beam details

The prestressed concrete beams were cast in Megaton (Willy Naessens Group) at the end of 2019 using the Hoyer method. The beam's cross-section is shown

in Figure 8-3. The length of each beam is 8 m, and the total volume of 1 beam is 1,272 m^3. The beams are reinforced with 9 prestressed strands, divided in the cross-section, as shown in Figure 8-4. The prestressing 7-wire strands have a diameter of 15.2 mm and a nominal cross-sectional area of 139 mm^2. The f_{pk} of the prestressing steel is 1860 N/mm^2, while the E_{pm} is 195000 N/mm^2.

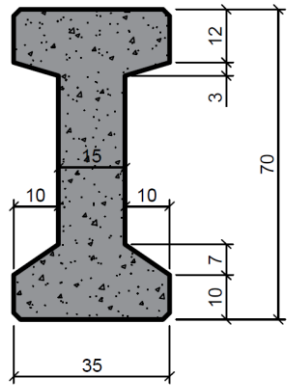

Figure 8-3 – Cross-section of the beam (dimensions in cm)

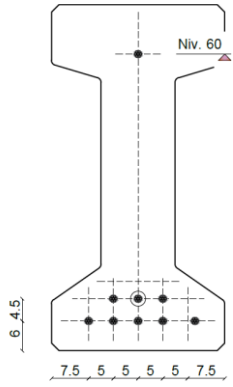

Figure 8-4 – Beam cross-section with prestressing strands

8.3.3 Prestress transfer

The initial prestress force is 200 kN/strand. Consequently, the initial prestress on the strands will be 1526 N/mm^2. Due to practical reasons, the prestress transfer is done at a concrete age of 3 days. The cylinder compressive strength at the prestress transfer moment is measured and is 61.28 MPa, 58.75 MPa and 56.78 MPa for RASCC0, RASCC20 and RASCC100, respectively.

The prestressing force to the concrete beam is in the case of prestressed concrete transferred on a specific length through shear forces between the concrete and the prestressing strands. The spread from a local concentrated force distribution to a linear force distribution happens along the anchorage length. The additional tensile forces in the ends of the beam are compensated by extra reinforcement. However, the anchorage length is in most cases limited and not considered in the calculation of the camber [167,169,171–173,175]. For further calculations in this chapter, a constant bending moment is assumed along the total length of the beam.

8.3.4 Test procedure

An industrial mixture is tested with different RCA replacement percentages in these test series. Each mixture is cast in a total of 1 prestressed beam, 45 cubes and 12 cylinders. The analysis of the tests on the cubes and cylinders is already discussed in Chapter 7. This chapter discusses the camber of the prestressed concrete beams.

The vertical displacement due to the release of the prestress force and the self-weight of the beams is measured at different moments: before the prestress transfer, immediately after the prestress transfer and at 1, 2, 4, 7, 14, 21, 25, 42, 60, 73, 103, 158, 179, 216, 266, 292, 329, 365 and 399 days after the prestress transfer. The camber is measured with a displacement meter to measure the deflection magnitude with high accuracy. Different displacement meters, shown in Figure 8-5, are placed in the middle of the beams to measure the deflection at the moments mentioned above.

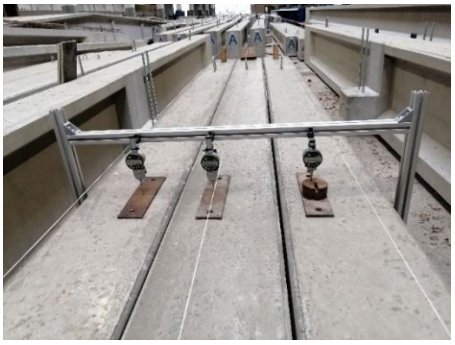

Figure 8-5 – Test setup

8.4 Initial camber

The initial camber occurs when the prestressing force is released to the concrete member. The tension forces in the prestressed strands transfer to the surrounding concrete, the bond between the strands and the concrete creates a compressive stress in the beam. As a result, a constant bending moment is developed along the length of the beam, yielding the camber. It is essential that the initial camber is predicted and measured accurately because it is often needed to predict the long-term camber of the elements [162]. Nevertheless, an inaccurate camber prediction does not affect the load-carrying capacity of a beam [175].

Most existing models for calculating the long-term camber of prestressed concrete members are based on the initial camber. Inaccuracies associated with the initial camber contribute in this way to an inaccurate evaluation of the long-term camber. The inaccuracies can be due to inaccurate assumptions in the design process, such as material properties, or different parameters affecting the measuring accuracy, such as bed frictions, bed deflections and human errors [162].

8.4.1 Factors influencing the initial camber

The camber of prestressed concrete beams is affected by different factors such as the prestress force and the concrete elasticity modulus. A review of the factors that influence the initial camber predictions of prestressed concrete members is presented in this section.

8.4.1.1 Prestress losses

The initial prestressing force in the prestressed beams is reduced during the curing time and service life of the beams. The effective prestressing force is defined as the initial prestressing force subtracted by the prestress losses. The prestress losses can be split into initial losses and long-term losses. The initial prestress losses occur when the prestress force transfers from the strands to the concrete beams, initiated by elastic shortening of the beams and anchorage losses [1]. When the prestress force is transferred to the concrete beam, it causes an elastic shortening of the beam. Since the prestressed strands are bonded with the surrounding concrete, a shortening of the concrete also implies a shortening of the prestressing strands. This phenomenon causes a loss in prestressing force in the concrete beams [162].

8.4.1.2 Concrete properties – modulus of elasticity

In practice, the concrete modulus of elasticity is often estimated based on the concrete compressive strength. The actual compressive strength is, in most cases, higher than the adopted compressive strength used in the design of the prestressed concrete. Therefore, it is evident that differences between the estimated and actual compressive strength will be reflected in differences between estimated and actual elastic modulus. These differences in elastic modulus of the concrete will also result in a difference between the estimated and actual camber. O'Neill et al. [174] reported that the actual compressive strength for concrete collected from 2006-2010 is on average 15.5% higher than the estimated compressive strength at the moment of prestress transfer. Since the compressive strength is used to predict the elastic modulus, significant

differences between the actual and design values of the concrete compressive strength also significantly affect the initial camber calculations.

The moment for the prestress transfer is influenced by the minimum strength requirements for concrete and the practical situation in the precast plant. In this study, the prestress could only be transferred three days after the prestressed concrete beams were manufactured on a Friday. A comparison between the estimated values for this type of concrete and the actual measured values of the reference concrete is shown in Table 8-1. The concrete compressive strength at the release of the prestress $f_c(t)$ could be estimated by the model in Eurocode 2 based on the targeted strength class of C50/60.

A strength class C50/60 corresponds to a characteristic cylinder compressive strength of 50 MPa at the age of 28 days, resulting in a mean compressive strength of 58 MPa according to Eurocode 2 [1]. The 3d-compressive strength of the concrete is calculated using the formulas 3.1 and 3.2 from Eurocode 2 [1]. The elastic modulus is calculated based on Eq.(4-10) in Chapter 4. The measured value is an average value of three cylinders, tested at the moment of the prestress transfer, three days after the concrete production.

The results in Table 8-1 show that the measured values for the compressive strength and modulus of elasticity of the concrete are remarkably higher than the estimated ones. This will not have structural consequences since higher strength and higher elastic modulus will result in a lower camber value. Nevertheless, the estimated camber value can be much larger than the actual camber value due to significant discrepancies between the estimated values and the measured values of the compressive strength and elastic modulus. In this regard, it is necessary to measure the actual compressive strength at the moment of prestress transfer to better estimate the initial camber of structural members.

Table 8-1 – Comparison between design values of RASCC0 and actual measured values of RASCC0

	Estimated value RASCC0	Measured value RASCC0
f_{cm}(3 days)	38.5 MPa	61.3 MPa
E_{cm}(3 days)	29331 MPa	37310 MPa

8.4.2 Measurements

The initial camber is measured immediately after the prestressing force is transferred to the concrete beam. It should be noted that before the prestress transfer, the beam is supported along its entire length by the prestressing bed, as shown in Figure 8-6. After the transfer of the prestressing force to the concrete beam, the vertical deflection or initial camber is measured. The initial camber of the three beams is for RASCC0, RASCC20 and RASCC100, respectively, 8.45, 8.70 and 9.25 mm. From the results can be concluded that the presence of RCA increases the initial camber of the prestressed concrete beams.

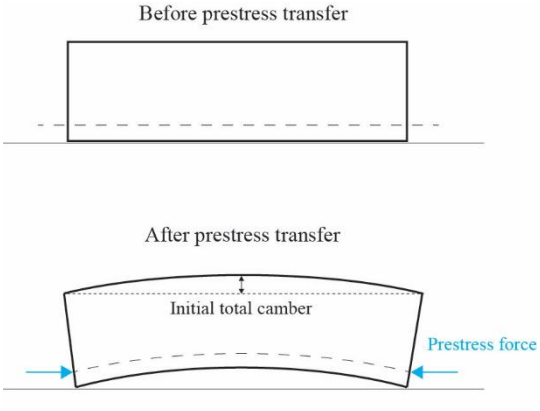

Figure 8-6 – Measuring procedure for initial camber

8.4.3 Calculation of the initial camber

Due to the assumed linear elastic behaviour of the system, the conventional theory of elasticity and the method of superposition are valid [173]. Thus, the deflection due to self-weight is calculated separately from the deflection due to the initial prestress, though these two factors determining the initial camber cannot be physically separated. A correct representation of both the material properties and the prestressing force is crucial for accurately determining the total camber of a precast concrete beam.

8.4.3.1 Elastic modulus

The elastic modulus at the moment of the prestress transfer, in this case at three days, can be determined in two ways. Cylinders can be poured with the same mixture as the concrete beams and consequently tested on the elastic modulus. However, this method is not used on an industrial level due to the advanced testing equipment necessary for testing the elastic modulus. The compressive strength of the concrete mixture can be easily determined and used to calculate the elastic modulus of the concrete by Eq.(4.10) from ACI 209 [2]. The difference between the measured and predicted value is relatively small. The measured values are used in these calculations.

Table 8-2 - Calculation of elastic modulus

	f_{cm}(3d) (MPa)	$E_{cm,measured}$(3d) (MPa)	$E_{cm,predicted}$(3d) (MPa)
RASCC0	61.28	37310	37018
RASCC20	58.75	36202	36255
RASCC100	56.78	35785	35642

8.4.3.2 Prestress force

The initial prestress force applied on the prestress strands before pouring the concrete beam is 200 kN for each strand. The prestressing force is distributed over the 9 prestressing strands, resulting in a total initial prestress force of 1800

kN. The prestress force is transferred three days after the concrete pouring. The prestress losses due to anchorage-loss and elastic shortening of the concrete are considered in this study.

(1) The anchorage losses occur due to slipping of the anchorage blocks next to the prestress bed. The losses are calculated according to Eq.(8-2):

$$\Delta f_{anchor} = \text{Slip x } \frac{E_p}{\text{Length of the cable}} \qquad (8\text{-}2)$$

A standard slip of 2 mm is assumed on every bed in this study, based on previous test experiences. The beams in this research were made on a concrete bed of 110 m. The E_p is 195000 N/mm^2. This results in an anchorage loss of 3.55 N/mm^2. The loss of prestressing force over the 9 strands is therefore 4.4 kN.

(2) The elastic shortening occurs at the time of prestress transfer due to the shortening of the beam. This loss of prestressing force can be estimated by Eq.(8-3) [172]:

$$\Delta f_{elastic} = \frac{E_p}{E_c} f_c \qquad (8\text{-}3)$$

Where:

E_p = elastic modulus of prestressing strands = *195000 MPa*

E_c = elastic modulus of the precast concrete at the release of the prestressing force, as shown in Table 8-2.

f_c = stress in the concrete at the centre of the prestressing strands after the prestress transfer due to prestressing and self-weight of the beam

$$= \left(\frac{P_0}{A_b} + \frac{P_0 e^2}{I_b}\right) - \frac{M_b e}{I_b} = 20.05 \text{ MPa}$$

P_0 = initial prestressing force just before the release of the prestress = *1795565 N*

e = eccentricity of the prestressing strands at midspan = *215 mm*

A_b = area of the gross cross-section of the beam = *159000 mm²*

I_g = moment of inertia of the beam = 8679050000 mm^4

M_b = moment at midspan due to the self-weight of the prestressed concrete beam = *31920000 Nmm*

The total prestress losses due to elastic shortening of the beam with RASCC0, calculated by the above equations, is 104.9 N/mm^2, corresponding to 131.2 kN spread over the 9 strands. The prestressing force applied to the beam is immediately diminished with the losses calculated in the paragraphs above. The resultant prestressing force for the RASCC0 beam is calculated as follows:

$$P_i = 1800 \text{ kN} - 4.4 \text{ kN} - 131.2 \text{ kN} = 1664.3 \text{ kN}$$

The initial prestressing forces immediately after the prestress transfer for the RASCC20 and RASCC100 beams are 1660.4 kN and 1658.8 kN, respectively.

8.4.4 Deflection due to the prestressing force

The initial camber of the prestressed concrete beam caused by the initial prestressing force can be calculated according to Eq.(8-4) [173]:

$$\Delta_{ip} = \frac{P_i \, e \, L^2}{8 \, E \, I} \qquad (8\text{-}4)$$

- P_i = Initial prestressing force in the group of strands, reduced by the immediate prestress losses, at the moment of prestress transfer (kN)
- e = Eccentricity of the prestressing tendons (mm)
- L = Span of the beam (mm)
- E = Elastic modulus of the concrete (N/mm^2)
- I = Moment of inertia (mm^4)

Table 8-3 – Upward deflection due to prestress force

	RASCC0	RASCC20	RASCC100
Moment of inertia (mm^4)	8679050000	8679050000	8679050000
Eccentricity (mm)	215	215	215
Prestress force (N)	1664371	1660355	1658780
Modulus of elasticity (N/mm^2)	37310	36202	35785
Span (mm)	8000	8000	8000
Deflection due to prestress (mm)	8.84	9.09	9.19

8.4.5 Deflection due to the self-weight of beams

The downwards deflection due to the self-weight of the beams can be calculated according to Eq.(8-5):

$$\Delta_g = \frac{5\ W\ L^4}{384\ E\ I} \qquad (8\text{-}5)$$

- W = self-weight of the beam (N/mm)
- L = Span of the beam (mm)
- E = Elastic modulus of the concrete (N/mm^2)
- I = Moment of inertia (mm^4)

Table 8-4 - Downwards deflection due to own weight

	RASCC0	RASCC20	RASCC100
Moment of inertia (mm^4)	8679050000	8679050000	8679050000
Own weight (N/mm)	3.99	3.99	3.96
Modulus of elasticity (N/mm^2)	37310	36202	35785
Span (mm)	8000	8000	8000
Deflection due to own weight (mm)	0.66	0.68	0.68

8.4.6 Total initial camber

The total initial camber can be calculated by subtracting the downward deflection from the upward deflection of the concrete beam. The calculated camber is for the prestressed concrete beam of RASCC0, RASCC20 and RASCC100, respectively 8.18, 8.41 and 8.51 mm. A comparison of the measured initial camber of the three beams is shown in Figure 8-7. The measured initial camber is generally higher than the calculated initial camber, 3%, 3% and 9% for RASCC0, RASCC20 and RASCC100, respectively. The increased initial camber for the RASCC100 compared to RASCC0 is mainly caused by the lower elastic modulus of RASCC100.

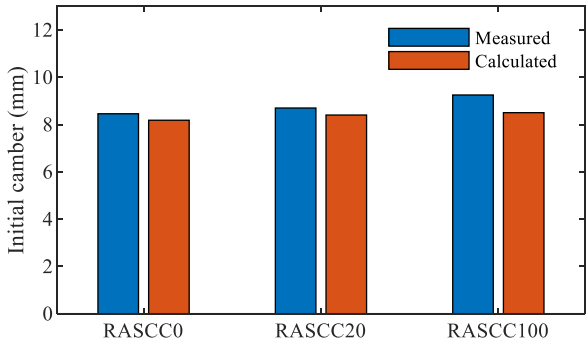

Figure 8-7 - Initial camber - measured vs calculated

8.5 Long-term camber

After the initial deflection due to the prestressing force and the own weight of the beam, several measurements at different time intervals were performed to determine the long-term camber of the prestressed concrete beams. The long-term camber is defined as the total camber subtracted by the initial camber of the prestressed beam. Since the long-term camber of the prestressed concrete beam is affected by the material properties of concrete, there is a significant possibility that the RCA influence the long-term camber in the mixtures. The long-term camber of the prestressed concrete beam is generally affected by more factors than the initial camber, including the concrete time-dependent properties, prestress losses and thermal effects. Due to the uncertainty of these parameters, it is very complicated to accurately predict the long-term camber of prestressed concrete beams. Consequently, the difference between the predictions and the actual measurements can be quite large, which can be an important problem concerning the requirements of prestressed beams in the serviceability state. Variations in the camber between different beams or inaccurate camber predictions will result in problems with the connection of the beams to roof panels or other beams.

8.5.1 Parameters affecting the long-term camber

8.5.1.1 Creep

Creep is a time-dependent effect that plays a significant role in the development of the camber of prestressed concrete beams. The best way to include concrete creep in models for camber predictions is by measuring the creep of the exact concrete mixtures used in the prestressed beams. This is rather time-consuming, so estimations and prediction models for creep are often used. The most commonly used models for predicting the concrete creep are the ones given in AASHTO LRFD Bridge Design Specifications [185], ACI 209R [186], *fib* Model Code 2010 [82], Eurocode 2 [1] and B3 model by Bazant [187]. Further calculations in this PhD research with concrete creep are carried out using *fib* Model Code 2010 [82].

The incorporation of RCA can significantly influence the creep of concrete. Lye et al. [188] performed a thorough literature study on the creep of concrete incorporating RCA. The authors concluded that the vast majority of the studies showed that the use of RCA negatively influences the creep of concrete. 70% of the investigated studies concluded an increased concrete creep, caused mainly by the adherent mortar, higher porosity, and lower elastic modulus of the used RCA. Van Hyfte et al. [189] also investigated the results of many studies on the influence of RCA on the creep of concrete. It was found that RAC generally has a higher creep deformation than reference concrete without RCA, with an average increase of 37.3% when the coarse aggregate is entirely replaced with RCA. High-quality RCA is most likely to have a lower increase in creep, while low-quality RCA might have a higher increase in concrete creep. Brandes and Kurama [39] investigated the influence of high-quality RCA on the creep and shrinkage strains of concrete cylinders. It was clear that the shrinkage strain and creep strain of the mixtures increased when high-quality RCA were incorporated into the concrete mixtures. The increase of the shrinkage and creep of RAC relative to the reference concrete with natural aggregates showed a considerable variation during the early stages but stabilised during the later stages of the tests. Their results showed an increase of 10-20% of the creep coefficient for RAC, compared to that of the reference concrete.

8.5.1.2 Prestress losses

The initial prestressing force is affected by different time-dependent factors, such as the creep and shrinkage of the concrete and relaxation of the strands. With the initial prestress losses, as discussed in Section 8.4.3.1, a total prestress loss gradually forms in time. This loss of prestress levels off towards an asymptotic value. Generally, 20-30% of the initial prestressing force is lost due to these factors. This has a pronounced effect on the camber of the prestressed concrete beams since the prestressing force is directly responsible for the camber deflection.

8.5.1.3 Thermal effects

Prestressed concrete beams are generally affected by thermal effects due to the variation in weather conditions. This depends on the way they are stored. Thermal changes due to the transportation and placement of the beams can induce changes in the camber. In this research, thermal changes have been minimised by storing the three beams with different mixtures in the same condition.

8.5.2 Measurements of long-term camber

The long-term deflections at the midspan of the prestressed concrete beams were measured at different moments to map the development of the camber for up to 399 days. The ambient temperature and relative humidity (RH) were also measured each moment when the camber deflections of the beams were measured, as shown in Figure 8-8 and Figure 8-9.

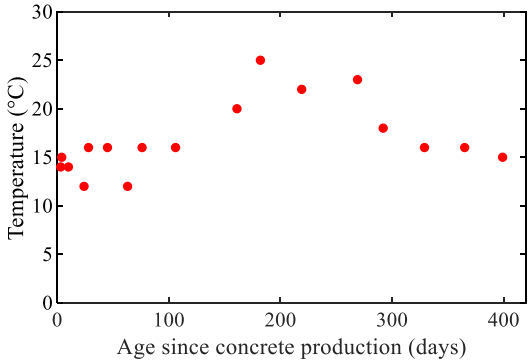

Figure 8-8 – Ambient temperature during storage of the concrete

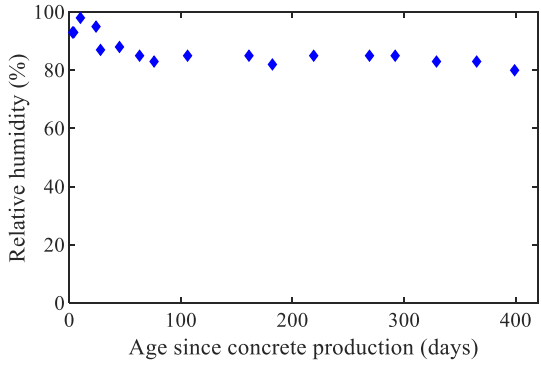

Figure 8-9 – Relative humidity during storage of the concrete

The absolute values of the upward deformation of each beam versus time are plotted in Figure 8-10. The incorporation of RCA into the prestressed concrete beams results in an increase in camber values. At the release of the prestressing force, the beam with RASCC100 already had the highest initial camber, as discussed above. The difference between the camber of the beam with RASCC100 and that of the beams with the mixtures RASCC0 and RASCC20 slightly enlarges over time.

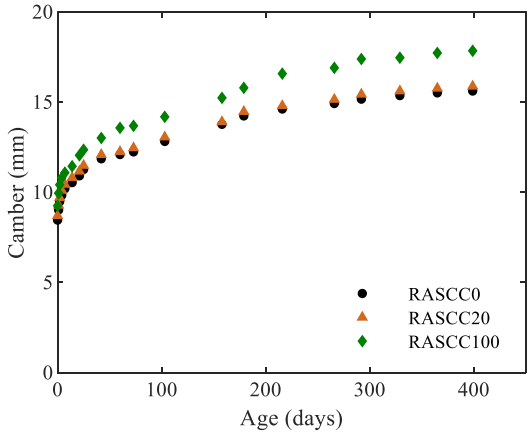

Figure 8-10 - Camber evolution of the three prestressed beams

8.5.3 Prediction methods of long-term camber

Various methods to calculate the long-term camber behaviour of prestressed concrete beams have been developed in the past years. Some include a detailed calculation using the mix design, concrete creep coefficient, drying shrinkage formulas and environmental parameters [190–193]. The problem with these types of calculations is the complexity in practical use for designers. Even with this detailed calculation, the accuracy of the predictions cannot be confirmed [176]. Using the finite element method, specialised software can predict the long-term behaviour of prestressed concrete beams [194–197]. However, for designers, a simple and straightforward prediction method is preferable. In multiplier methods, suitable multipliers are applied to the initial camber or the components of the initial camber (i.e., initial camber caused by the prestressing force and self-weight) to predict the camber at placement. It has been customary in daily design to estimate the long-term camber of the prestressed concrete members by multiplying the initial camber Δ_i by an appropriate factor. The basic idea behind this approach originates from the analysis of the effect of creep strain under initial loading, assuming that a test sample is subjected to a permanent load. The creep strain $\varepsilon_{cC}(t)$ can be calculated by multiplying the initial strain ε_{ci} by the creep coefficient $C_C(t)$, as shown in Figure 8-11 and Eq.(8-6).

$$\varepsilon_{cC}(t) = \varepsilon_{ci} \times C_C(t) \tag{8-6}$$

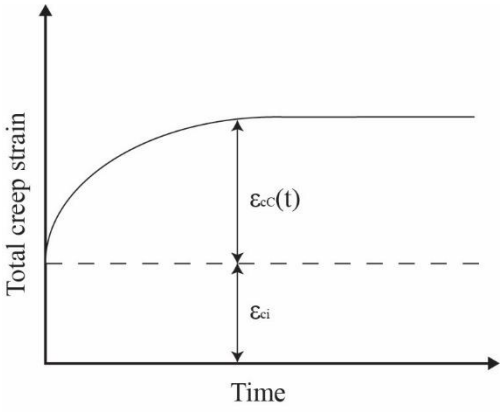

Figure 8-11 – Creep strain in time

In Eq.(8-6), the multiplier to estimate the creep deflection is defined as the creep coefficient. For the estimation of the camber, other parameters to estimate the long-term camber of prestressed beams, like the effect of shrinkage, prestress loss, and others should be included. In analogy with creep, the total camber of prestressed beams can be calculated according to Eq.(8-7) and Eq.(8-8) by using the multiplier $\lambda(t)$.

$$\Delta_{lt}(t) = \lambda(t)\ \Delta_i \tag{8-7}$$

$$\Delta(t) = \Delta_i\ (1+ \lambda(t)) \tag{8-8}$$

The exact time of the placement of the prestressed concrete beams on the construction site is, in most cases, unknown. The uncertainty can result in a reduction in the accuracy of the camber prediction. Therefore, making the multiplier $\lambda(t)$ time-dependent can increase the accuracy. Gheitanbaf [176] simplified the calculation by proposing fixed multipliers for several time intervals. These time intervals are 0-60 days, 60-180 days and 180-480 days. The corresponding multipliers to the mentioned time intervals are 1.53 ± 0.02, 1.61 ± 0.02 and 1.67 ± 0.02, respectively. The increase in camber later than 480 days is expected to be minor. The difference between the actual camber and the estimations using the fixed multipliers might be large since the fixed time

intervals are not ideal for accurate estimations. For practical applications, a multiplier as a function of time is preferred to determine an accurate prediction of the camber at the exact time of the placement of the concrete beams [162]. The multiplier for beams with no overhang in Eq.(8-9) is calculated as a power function of time for prestressed beams, where t is expressed in days.

$$\Delta_{add}(t) = 1.264 \text{ x } t^{0.045} \text{ [176]} \qquad (8\text{-}9)$$

Multipliers with only time as a variable parameter can be used for the same concrete mixture cured in the same conditions. Since creep and shrinkage of concrete depend on the ambient conditions and properties of the mixture, it is recommended to include variable parameters covering prestress losses and concrete creep. The following two methods are introduced to predict the ultimate long-term deflection of prestressed concrete members considering prestress losses, elastic modulus, and creep coefficient.

8.5.3.1 Long-term camber methods with multipliers

Branson et al. [163] proposed Eq.(8-10) to predict the long-term camber Δ_{lt} in conventional prestressed concrete members:

$$\Delta_{lt} = \left(\eta\text{-}1+\left(\frac{1+\eta}{2}\right)k_r C_{CU}\right)(\Delta_i)_{Fi}+(k_r C_{CU})(\Delta_i)_G \qquad (8\text{-}10)$$

Where

- $(\Delta_i)_{Fi}$ is the initial camber due to the prestressing force
- $(\Delta_i)_G$ is the initial deflection due to the self-weight of the beams
- $\eta = F/F_i$ with F_i is the initial prestressing force on the moment of the prestress transfer – F is the final prestressing force
- $k_r = 1/(1+A_s/A_{ps})$ with A_s the area of the tensile non-prestressing reinforcement and A_{ps} the area of the prestressing reinforcement
- C_{CU} is the ultimate creep coefficient

Based on an extensive evaluation of parameters influencing the long-term deflection of prestressed concrete beams at service state, Martin [166] proposed the following equation:

$$\Delta_{lt} = \left(\eta\left(\frac{E_{ci}}{E_c}\right)k_r C_{CU}\right)\Delta_i \qquad (8\text{-}11)$$

Where

- Δ_i is the initial deflection of the prestressed beam
- E_{ci} is the elastic modulus of the concrete at the moment of prestress transfer
- E_c is the final elastic modulus of the concrete
- The other parameters are similar to Eq.(8-10)

8.5.4 Validation of prediction methods for the long-term camber

8.5.4.1 Assumptions

Ambient conditions

The exact ambient conditions of the prestressed concrete beams were not constant since the beams were not stored in a climate room but in the uninsulated storage room of the precast plant. The temperature and relative humidity changed with weather condition variations. An average value for the ambient temperature of 18.1°C and relative humidity of 84.7%, calculated throughout all the measurements in Figure 8-8 and Figure 8-9, is assumed in the following calculations.

Creep coefficient

The creep coefficient is calculated according to the method described in the *fib* Model Code 2010, using different equations as shown in Table 8-6 [82]. The creep coefficient values in Table 8-5 and Figure 8-12 are used for further calculations. The ultimate creep coefficient is calculated based on a period of 50 years.

Table 8-5 – Creep coefficient of different mixtures

	RASCC0	RASCC20	RASCC100
t (days)	399	399	399
t_0 (days)	3	3	3
f_{cm} (MPa)	78.3	75.5	79.9
$\varphi(t,t_0)$	0.91	0.93	0.91
$\varphi(\infty,t_0)$	1.28	1.31	1.28

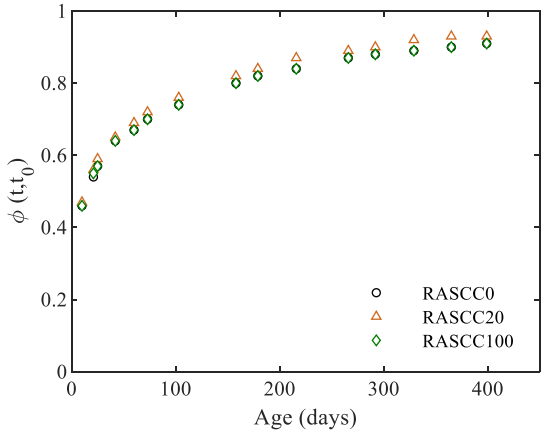

Figure 8-12 – Development of creep coefficient according to fib Model Code 2010 [82]

Table 8-6 – Calculation of creep coefficient $\varphi(t,t_0)$ according to fib Model Code 2010 [82]

$$\varphi(t,t_0) = \varphi_{bc}(t,t_0) + \varphi_{dc}(t,t_0)$$

$$\varphi_{bc}(t,t_0) = \beta_{bc}(f_{cm}) \times \beta_{bc}(t,t_0)$$

$$\beta_{bc}(f_{cm}) = 1.8 / (f_{cm})^{0.7}$$

$$\beta_{bc}(t,t_0) = \ln((30/t_{0,adj} + 0.035)^2 \times (t - t_0) + 1)$$

$$\varphi_{dc}(t,t_0) = \beta_{dc}(f_{cm}) \times \beta(RH) \times \beta_{dc}(t_0) \times \beta_{dc}(t,t_0)$$

$$\beta_{dc}(f_{cm}) = 412/(f_{cm})^{1.4}$$

$$\beta(RH) = (1 - 0.01RH)/(0.1 \times 0.01h_0)^{1/3}$$

$$\beta_{dc}(t_0) = 1/(0.1+t_{0,adj}^{0.2})$$

$$\beta_{dc}(t,t_0) = \frac{(t-t0)}{\beta_h+(t-t_0)}^{\gamma(t_0)}$$

$$\gamma(t_0)=\frac{1}{2.3+\frac{3.5}{\sqrt{t_{0,adj}}}}$$

$$\beta_h = 1.5\ h + 250\ \alpha_{fcm} \leq 1500\ \alpha_{fcm}$$

$$\alpha_{fcm} = (35/f_{cm})^{0.5}$$

$$t_{0,adj}= t_{0,T}\ x\ \left(\frac{9}{2+t_{0,T}^{1.2}}+1\right) \text{ for CEM I 52.5R}$$

$$t_{0,T}= t_0\ x\ \exp\left(13.65-\frac{4000}{273+T_0}\right)$$

With

- $\varphi_{bc}(t,t_0)$ = basic creep coefficient
- $\varphi_{dc}(t,t_0)$ = drying creep coefficient
- t = age of concrete at the moment considered (days)
- t_0 = age of concrete at loading (days)
- $t_{0,adj}$ = adjusted age of concrete at loading according to cement-type (days)
- $t_{0,T}$ = adjusted age of concrete at loading according to temperature (days)
- RH = relative humidity (%)
- h_0 = notional height of concrete member (m)

Prestress losses

The prestress losses include the immediate prestress losses discussed in Section 8.4.3.2 and time-dependent losses due to creep and shrinkage of concrete as well as the relaxation. The time-dependent losses are estimated according to the calculated creep and shrinkage of concrete as well as the relaxation losses in *fib* Model Code 2010 [82].

➔ The concrete beam experiences a shortening under the constant load of the prestress force. The horizontal shortening of the prestressed beam induces a prestress loss, as the prestressed strands are embedded in the concrete. The prestress losses due to the creep deformation $\Delta f_{p,cr}$ is calculated according to Eq.(8-12). The development of this type of prestress losses is similar to the creep coefficient, as shown in Figure 8-13.

$$\Delta f_{p,cr} = E_p \varepsilon_{cr} \qquad (8\text{-}12)$$

Where:

$\Delta f_{p,cr}$ = Prestress losses due to creep strain

E_p = Elastic modulus of prestressing strands

ε_{cr} = Creep strain calculated by C_c * (f_c/E_c)

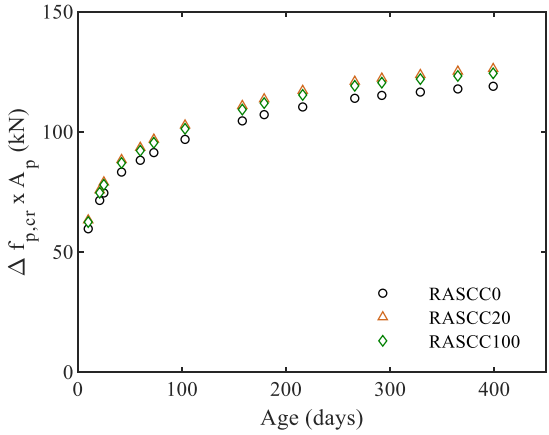

Figure 8-13 – Development of prestress losses due to creep of concrete

➔ Due to the development of shrinkage in the concrete beams, a shrinkage strain induces extra prestress losses. The prestress losses due to shrinkage are calculated according to Eq.(8-13).

$$\Delta f_{p,sh} = E_p \varepsilon_{sh} \tag{8-13}$$

Where:

$\Delta f_{p,sh}$ = Prestress losses due to shrinkage strain

ε_{sh} = Shrinkage strain calculated according to *fib* Model Code 2010 [82] and shown in Table 8-7. The shrinkage strain is calculated by the calculation method in Table 8-8.

Table 8-7 – Shrinkage strain calculated according to fib Model Code 2010 [82]

t (days)	ε_{sh} RASCC0	ε_{sh} RASCC20	ε_{sh} RASCC100
10	-0.000084	-0.000082	-0.000083
21	-0.000112	-0.000110	-0.000111
25	-0.000119	-0.000117	-0.000119

42	-0.000143	-0.000141	-0.000143
60	-0.000160	-0.000158	-0.000160
73	-0.000170	-0.000168	-0.000170
103	-0.000188	-0.000186	-0.000188
158	-0.000211	-0.000209	-0.000211
179	-0.000218	-0.000215	-0.000217
216	-0.000227	-0.000225	-0.000227
266	-0.000238	-0.000236	-0.000238
292	-0.000243	-0.000241	-0.000243
329	-0.000249	-0.000247	-0.000249
365	-0.000254	-0.000252	-0.000254
399	-0.000258	-0.000257	-0.000258
final	-0.000351	-0.000352	-0.000351

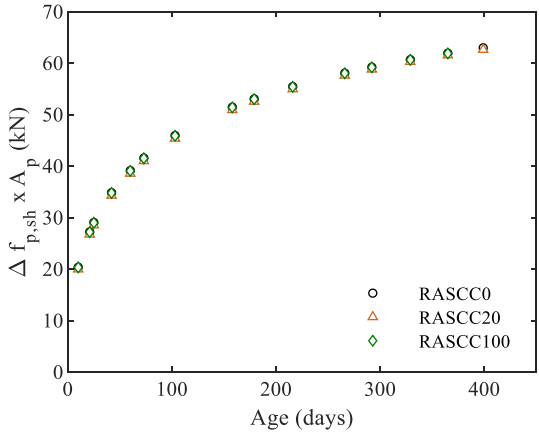

Figure 8-14 – Development of prestress losses due to shrinkage of concrete

Table 8-8 – Calculation of shrinkage strain $\varepsilon_{cs}(t,t_s)$ according to fib Model Code 2010 for concrete made with CEM I 52.5R [82]

$\varepsilon_{cs}(t,t_s) = \varepsilon_{cbs}(t,t_s) + \varepsilon_{cds}(t,t_s)$
$\varepsilon_{cbs}(t,t_s) = \varepsilon_{cbs0}(f_{cm}) \times \beta_{bs}(t)$
$\varepsilon_{cbs0}(f_{cm}) = -600 \left(\frac{0.1\, f_{cm}}{6+0.1 f_{cm}}\right)^{2.5} 10^{-6}$

$$\beta_{bs}(t) = 1 - \exp(-0.2\sqrt{t})$$

$$\varepsilon_{cds}(t,t_s) = \varepsilon_{cds0}(f_{cm}) \times \beta_s(RH) \times \beta_{ds}(t-t_s)$$

$$\varepsilon_{cds0}(f_{cm}) = (220+110x6) \times \exp(-0.012\, f_{cm}) \times 10^{-6}$$

$$\beta_s(RH) = -1.55 \times (1-0.01RH)^3$$

$$\beta_{ds}(t-t_s) = \left(\frac{(t-t_s)}{0.035h_0^2+(t-t_s)}\right)^{0.5}$$

With

- $\varepsilon_{cbs}(t,t_s)$ = basic shrinkage
- $\varepsilon_{cds}(t,t_s)$ = drying shrinkage
- t = concrete age (days)
- t_s = concrete age at the beginning of drying (days)
- RH = relative humidity (%)
- h_0 = notional height of a concrete beam

➔ Relaxation of the prestressed strands occurs by the reduction of the stress in the prestressed strands due to a constant applied strain. The prestress losses due to the relaxation are calculated according to *fib* Model Code 2010 [82].

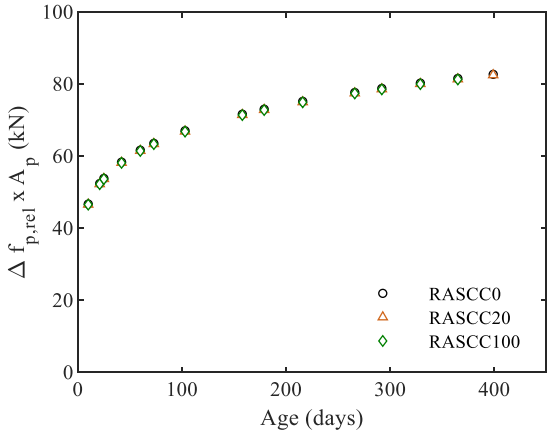

Figure 8-15 – Development of prestress losses due to relaxation of strands

➔ The total prestress force is shown in Figure 8-16. The initial prestress force in the prestress beams is 1800 kN. The immediate prestress losses reduce the prestress force to 1664 kN for the RASCC0 beam. During the ageing period of the beams, the prestress losses due to concrete creep and shrinkage, as well as the relaxation, further reduce the prestressing force, as shown in Figure 8-16.

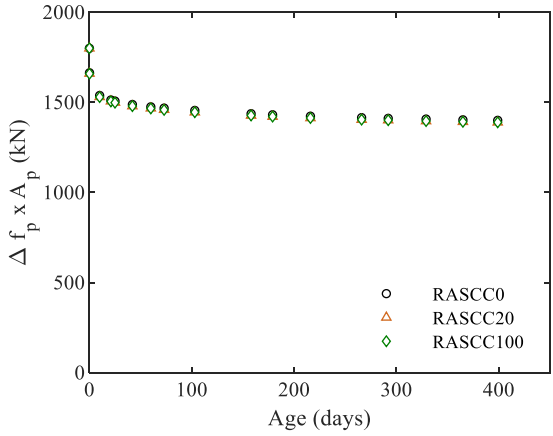

Figure 8-16 – Evolution of prestressing force in time

Initial camber

The initial camber is split into an upward and downward deflection to calculate the long-term deflection using Branson's multipliers [163]. The calculation of the initial prestress is divided into an upward and downward component, but the exact measurement is the combination of the two types of deflections and cannot be measured separately. As the initial camber measurement results differ slightly from the calculations in Section 8.2.2, the ratio of the calculated upward to the total camber and downward to the total camber is used to divide the measured initial camber into an upward and downward component.

Table 8-9 – Upward and downward component of the initial deflection

	RASCC0	RASCC20	RASCC100
$(\Delta_i)_{Fi,calc}$ (mm)	8.84	9.08	9.19
$(\Delta_i)_{G,calc}$ (mm)	0.66	0.68	0.69

$(\Delta_i)_{Fi,calc}$ / $\Delta_{i,\ calc}$	108%	108%	108%
$(\Delta_i)_{Fi,calc}$ / $\Delta_{i,\ calc}$ (%)	8%	8%	8%
$\Delta_{i,\ measured}$ (mm)	8.45	8.70	9.25
$(\Delta_i)_{Fi,estimated}$ (mm)	9.13	9.40	9.99
$(\Delta_i)_{G,estimated}$ (mm)	0.68	0.70	0.75

8.5.4.2 Long-term camber using Branson's multipliers

The long-term camber of prestressed concrete beams can be calculated according to Branson's equation Eq.(8-10). The different parameters are described in Section 8.5.4.1. According to Branson's multipliers and the measured values, a comparison between the predicted camber at the age of 399 days is shown in Table 8-10.

Table 8-10 – Long-term camber using Branson's multipliers

	RASCC0	RASCC20	RASCC100
$\Delta_{lt,calc}$ (mm)	6.79	7.17	7.39
$\Delta_{lt,meas}$ (mm)	7.16	7.18	8.59
Difference	5%	0%	14%

The ultimate creep coefficient is used in Equation 8-10 to predict the final long-term camber. The creep coefficient can be used as a time-variable parameter to determine the development of the long-term camber. The calculated values in Figure 8-12 are used. A comparison between the prediction from Branson's multipliers and the measured values of the long-term camber at the different ages is presented in Figure 8-17. The long-term camber gives an accurate description of the long-term camber at 399 days for the beams with RASCC0 and RASCC20, while the measured values for the beam with RASCC100 are higher. An extra correction factor is necessary to consider the influence of incorporating RCA in the mixture. The calculated long-term camber does not accurately predict the measured long-term camber before 200 days. The description of this model in Naaman [163] suggests predicting the final value of

the long-term camber since the ultimate creep coefficient is used. The development of the long-term camber is not specified.

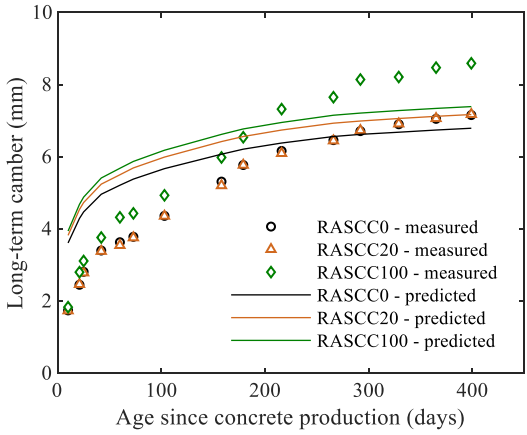

Figure 8-17 – Measured vs predicted long-term camber according to Branson's multiplier

8.5.4.3 Long-term camber using Martin's multipliers

Another calculation method to estimate the long-term camber is using Martin's multipliers. The multipliers are calculated according to Eq.(8-11). The calculated long-term camber according to Martin's multipliers is shown in Table 8-11. The creep coefficient as in Figure 8-12 is used as a variable parameter to calculate the development of the long-term camber, as shown in Figure 8-18. The prediction underestimates the measured long-term camber at 200-399 days, while it overestimates the measured long-term camber at an early age. Naaman [163] specified the calculation method of Martin for calculating the final value of the long-term camber since the ultimate creep coefficient is used. However, Martin [166] specifies the calculation method for different time intervals. Especially the time of beam placement on the construction site, which is an intermediate stage, is described as important. The beam placement can vary from 3 weeks to 4 months after the concrete production, depending on the construction schedule.

Table 8-11 – Long-term camber using Martin's multipliers

	RASCC0	RASCC20	RASCC100
$\Delta_{lt,calc}$ (mm)	5.50	5.65	5.79
$\Delta_{lt,meas}$ (mm)	7.16	7.18	8.59
Difference	23%	21%	33%

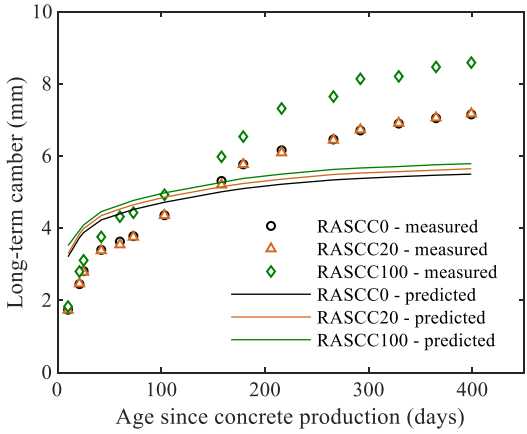

Figure 8-18 – Measured vs predicted long-term camber according to Martins' multiplier

8.5.4.4 Possible variations

The average temperature and relative humidity (RH) under which the prestressed concrete beams were cured, have been estimated based on regular measurements, as shown in Figure 8-8 and Figure 8-9. Nevertheless, since the storage rooms are not insulated, changes in weather conditions outside the room affect the ambient conditions in the storage room. Possible deviations from the estimated average values for the temperature and RH to their actual values can exist. Therefore, the influences of possible variations in temperature and RH have been assessed. The previous calculations have been conducted with average

values for RH and temperature of 84.7% and 18.1°C. The influence of possible variations in relative humidity is determined by assuming the extreme values of 95% as the upper limit of RH and 80% as the lower limit of RH. The temperature variation is determined by assuming a ±10% of the measured value, resulting in limits of 16.3°C and 19.9°C.

➔ The influence of the variations mentioned above on Branson's multipliers for the beam with RASCC0 is mentioned in Figure 8-19, Figure 8-20 and Table 8-12. From Figure 8-20 can be seen that the long-term camber does not vary much with a temperature variation. A variation in RH causes more considerable changes in the calculated long-term camber, as shown in Figure 8-19. An increase from the average RH to the maximum RH decreases the long-term camber by 10-13%, while a decrease from the average RH to the minimal RH increases the long-term camber by 4-6%. The measured long-term camber (>200 days) is comparable to the predicted long-term camber. It is clear that a decrease in the RH does not change the calculated long-term camber too much, but the increase of RH up to 95% leads to underestimating the final camber at the age of 399 days.

Table 8-12 – Influence of variation of RH and temperature on the calculation of the long-term camber of the beam with RASCC0 by Branson's multipliers

t (days)	$\Delta_{lt,calc}$ (mm)	$\Delta_{lt,calc}$ RH max (mm)	$\Delta_{lt,calc}$ RH min (mm)	$\Delta_{lt,calc}$ T+10% (mm)	$\Delta_{lt,calc}$ T-10% (mm)
10	3.61	3.26	3.77	3.61	3.62
21	4.29	3.85	4.49	4.28	4.30
25	4.47	4.00	4.68	4.47	4.48
42	4.96	4.41	5.20	4.96	4.95
60	5.21	4.62	5.48	5.21	5.21
73	5.38	4.76	5.66	5.38	5.38
103	5.67	4.99	5.98	5.67	5.66
158	6.07	5.32	6.42	6.08	6.06
179	6.21	5.43	6.57	6.23	6.19
216	6.38	5.56	6.75	6.40	6.36

266	6.56	5.71	6.94	6.58	6.54
292	6.62	5.76	7.01	6.64	6.59
329	6.68	5.81	7.08	6.70	6.66
365	6.74	5.86	7.14	6.76	6.72
399	6.79	5.90	7.19	6.80	6.77

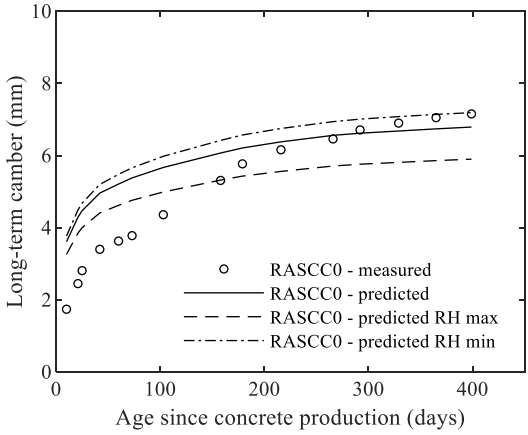

Figure 8-19 – Influence of variation of RH on the calculated long-term camber of the beam with RASCC0 by Branson's multipliers

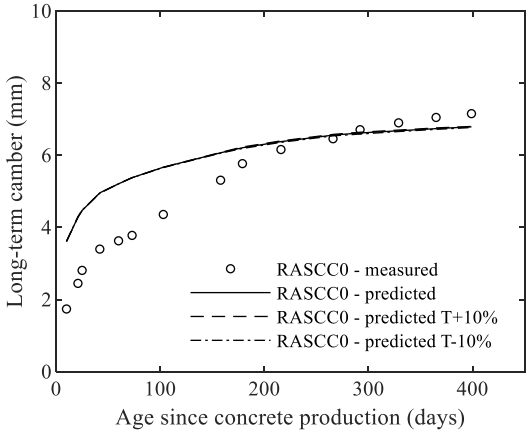

Figure 8-20 – Influence of variation of temperature on the calculated long-term camber of the beam with RASCC0 by Branson's multipliers

➔ The influence of the variations in the temperature and RH on the calculated long-term camber of the beams using Martin's multipliers for the beam with RASCC0 is mentioned in Figure 8-21, Figure 8-22 and Table 8-13. Similar to the calculated results using Branson's multipliers in the above section, the temperature variation does not influence the calculated long-term camber, while the RH variation considerably influences the calculations. An increase of the maximum RH results in a decrease in the long-term camber of 8-11%, while a decrease to the minimal RH increases the long-term camber of 4-5%. Even considering the possible variations in RH, the predictive calculations by means of Martin's multipliers still underestimate the measured long-term camber. The prediction at the moment of concrete placement can quite accurately describe the long-term camber of the prestressed concrete beams.

Table 8-13 - Influence of variation of relative humidity and temperature on the calculated long-term camber of the beam with RASCC0 by Branson's multipliers

t (days)	$\Delta_{lt,calc}$ (mm)	$\Delta_{lt,calc}$ RH max (mm)	$\Delta_{lt,calc}$ RH min (mm)	$\Delta_{lt,calc}$ T+10°C (mm)	$\Delta_{lt,calc}$ T-10°C (mm)
10	3.21	2.95	3.33	3.21	3.22
21	3.74	3.42	3.89	3.74	3.75
25	3.88	3.54	4.03	3.88	3.88
42	4.23	3.84	4.41	4.23	4.23
60	4.40	3.99	4.59	4.40	4.40
73	4.52	4.09	4.72	4.52	4.52
103	4.72	4.25	4.94	4.73	4.72
158	5.01	4.49	5.24	5.01	5.00
179	5.10	4.57	5.35	5.11	5.09
216	5.22	4.67	5.47	5.23	5.20
266	5.34	4.77	5.60	5.36	5.33
292	5.38	4.80	5.64	5.40	5.37
329	5.43	4.84	5.69	5.44	5.41

365	5.47	4.88	5.73	5.48	5.45
399	5.50	4.90	5.77	5.51	5.49

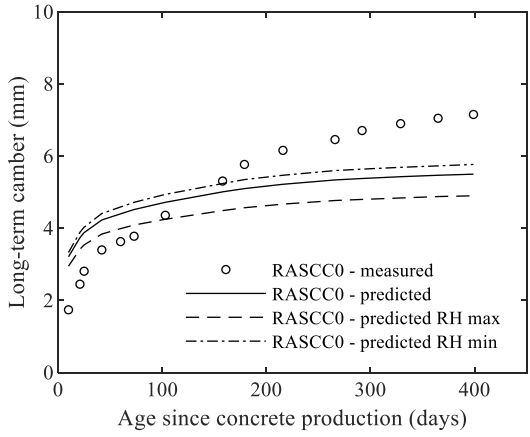

Figure 8-21 – Influence of variation of relative humidity on the calculated long-term camber of the beam with RASCC0 by Martin's multipliers

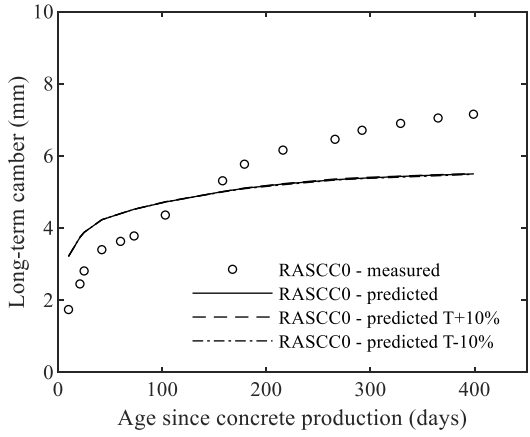

Figure 8-22 – Influence of variation of temperature on the calculated long-term camber of the beam with RASCC0 by Martin's multipliers

8.5.5 Influence of RCA on the relative camber

Figure 8-23 shows the relative camber, defined as the ratio of the total camber deformation to the initial camber. The increase of the relative camber for the beams with RASCC0 and RASCC20 in time are similar to each other. However, the RASCC100 beam reaches a relative camber of 1.93 at 399 days, while the RASCC0 and RASCC20 beams only reach a relative camber of 1.85 and 1.83, respectively.

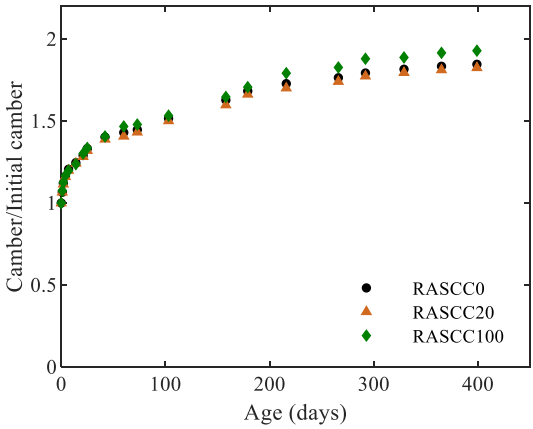

Figure 8-23 – Relative long-term camber

The effect of the presence of RCA in the concrete mixtures on the initial camber of the prestressed concrete beams has already been discussed in Section 8.4. In this section, the initial camber is excluded by zeroing the total camber right after the prestress transfer. In this way, only the long-term camber (the increment of the camber to the initial camber) is evaluated. The effect of incorporating RCA in the concrete on the long-term camber of the prestressed concrete beams can be defined by dividing the long-term camber of the beams with RASCC20 and RASCC100 by that of the reference beam with RASCC0 beam. From Figure 8-24 can be seen that the camber increase stabilises around a value of 1.2 for RASCC100. Thus, a complete replacement of the coarse fraction of limestone by high-quality RCA results in an increased long-term camber by 20%. A replacement of up to 20% does not seem to affect the

development of the long-term camber development of the prestressed concrete beams.

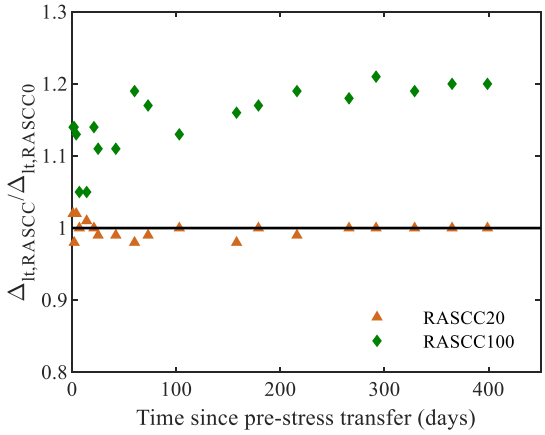

Figure 8-24 – Relative camber increase

Since the long-term camber of the prestressed concrete beams is primarily related to the time-dependent properties of the concrete, a direct relation with the creep deformation can be set. Previous studies indicated that incorporating RCA in concrete results in an increased creep, while the relative increase stabilises in time [39]. This phenomenon can also be noticed in Figure 8-24, where a relative long-term camber increase stabilises in time. This is in accordance with the results of Brandes and Kurama [39], who evaluated the service-load behaviour of precast prestressed concrete beams with RCA was evaluated. In their studies, the deflection amplifications were determined by dividing the deflections of the beams with RCA by that of the beams without RCA. The results revealed that the deflection amplifications stabilise after a certain period.

Several studies have tried to quantify the relative increase in concrete creep deformation due to the incorporation of RCA. Most previous studies concluded that a concrete creep increase of 30% to 60% is feasible, depending on the quality of the RCA [189]. Since creep is a concrete property frequently used in concrete design, some have proposed additional factors to include the difference in creep

behaviour for concrete with RCA. The Task Force of the Standing Committee of Concrete of Spain [198] proposed a correction factor of 1.00 for concrete with 20% replacement of coarse RCA. This correction factor is set on 1.25 to 1.45 for concrete with a complete replacement of the coarse fraction by RCA. EHE-08 proposed a similar correction factor of 1.25 for a complete replacement of RCA in concrete [199]. It is mentioned that this factor can change because of a changing quality of RCA. Replacement percentages lower than 20% should not result in an increased creep deformation.

Reducing the variability of RCA by using only high-quality RCA in structural concrete applications can improve the reliability of the correction factors. Previous attempts to develop a correction factor for creep related properties of RAC compared to NAC have some similarities with the results from the long-term camber, as shown in Figure 8-24. One could conclude from the results that concrete mixtures with a replacement up to 20 $\%_{V/V}$ of the coarse fraction of limestone do not require an additional correction factor for time-dependent properties, while a replacement of $100\%_{V/V}$ requires a correction factor of 1.20, as concluded from Figure 8-24. When the total coarse fraction (≥ 4mm) is taken into account, the RCA replacement percentage becomes 0, 14 and $71\%_{V/V}$, in the concrete mixtures in this study, respectively. When the total aggregates skeleton is considered, the RCA replacement percentages become 0, 5 and $27\%_{V/V}$. This is an essential aspect in considering any correction factors for the use of RCA in concrete since the replacement percentage of the total aggregate skeleton gives a better indication of the effect of RCA on the concrete properties.

8.6 Conclusions

This chapter presents a feasibility study using the high-quality RCA in pretensioned prestressed concrete beams on an industrial scale. The influence of different replacement percentages of RCA on the camber of the prestressed beams is measured and compared with different prediction models. Within the scope of this research, the following conclusions can be drawn.

The initial camber is the resultant deflection arising from the prestress transfer and the self-weight of the prestressed concrete beams. The measured initial

camber of the three beams RASCC0, RASCC20 and RASCC100, is 8.45, 8.70 and 9.25 mm, respectively. From the measured results can be concluded that the presence of RCA results in an increased initial camber of the prestressed concrete beams. The calculation of the initial camber is split up into the deflection due to the beams' self-weight and the deflection due to the initial prestress force, although these two factors determining the camber cannot be physically separated. The calculated camber for the prestressed beam with RASCC0, RASCC20 and RASCC100 is 8.18, 8.41 and 8.51 mm, respectively. It can be noticed that the measured initial camber is higher than the calculated initial camber, with an increase of 3%, 3% and 9% for the beams with RASCC0, RASCC20 and RASCC100, respectively. The increased initial camber for the beam with RASCC100 compared to that of the beam with RASCC0 is caused by the lower elastic modulus of RASCC100.

The long-term camber of all the three prestressed concrete beams is measured at different time intervals and shown in Figure 8-10. Two prediction methods, i.e., Branson and Martin, are used to predict the long-term deflection of the beams through multipliers. These multipliers are calculated based on different parameters such as the initial camber, prestress losses, creep coefficients and elastic modulus of concrete. Although these methods fail in accurately predicting the development of the long-term camber over the whole range, the final camber of the beams at the age of 399 days is accurately predicted by the equation of Branson.

The evaluation of the relative long-term camber of the prestressed concrete beams shows that a complete replacement of the coarse limestone fraction by the high-quality RCA increases the long-term camber by 20%. A replacement of the natural aggregates by the high-quality RCA up to $20\%_{V/V}$ does not seem to affect the long-term camber development of the prestressed concrete beams. One could conclude from the test results that concrete mixtures with a replacement up to 20 $\%_{V/V}$ of the coarse limestone fraction do not require an additional correction factor for calculating the long-term camber, while a complete replacement does require a correction factor of 1.20, as shown in Figure 8-24.

Chapter IX

Conclusions and perspectives

9.1 Conclusions

The main objective of this PhD project was to evaluate the feasibility of using RCA in precast structural concrete. The effect of high-quality RCA on different aspects of precast concrete was thoroughly investigated to have a clear view of the possibilities of using RCA in precast structural concrete. The early-age mechanical properties and the variability of two precast concrete mixtures were tested in laboratory conditions, together with a study on the bond behaviour between RAC and steel reinforcement. The laboratory tests were validated by repeating the lab tests on concrete produced in a precast plant, while three prestressed beams were made to assess the influence of the high-quality RCA on the camber of the precast concrete.

The novelty of this research is found in the combination of industrially produced high-quality RCA incorporated in industrial precast concrete mixtures. The industrial precast mixtures are high-strength concrete (C50/60) since this is frequently required in precast plants to achieve an adequate early-age compressive strength of concrete. A knowledge gap is noticed between the extensive research using normal lab-made RCA in normal strength concrete and actual practical situations. The practical use of high-quality RCA in precast structural concrete requires applied research with industrial conditions. The next steps in the actual use of high-quality RCA in precast products are only

possible by removing any restrictions concerning the use of RCA through systematic research.

9.1.1 Early-age mechanical properties of concrete with the RCA

Within the scope of this PhD research can be concluded that the effect of the high-quality RCA on the compressive strength of the tested mixtures is minimal. The compressive strength of precast structural concrete decreases 9% when the high-quality RCA replace 100% of the coarse limestone fraction. The strength decrease is negligible when the high-quality RCA replace the coarse limestone fraction in the self-compacting precast concrete mixtures, both in the laboratory and industrial tests. This is due to the smaller replacement percentage of the self-compacting precast concrete mixture when the complete aggregate skeleton is considered, as mentioned in Section 3.2.

A more accurate description of the strength development of the used precast concrete types can be achieved by adjusting the constants in Eq.(7-1) and Eq.(7-2) via the least-square method. The proposed constants in Table 7-6 accurately predict the strength development of the precast concrete mixtures, taking into account the incorporation of high-quality RCA and high-early strength cement. Incorporating the RCA in the mixtures slightly influences the strength development in the first days. A slower strength development for mixtures with high replacement percentages of the RCA is noticed in the first three days after concrete production. This effect can be minimised by limiting the amount of high-quality RCA to 20%$_{V/V}$ of the coarse limestone fraction.

The laboratory tests results show that the influence of the high-quality RCA on the tensile splitting strength and development is negligible. It is also clear that the relation between the compressive strength and the tensile splitting strength is not affected by incorporating high-quality RCA in precast concrete. Based on the regression analysis of the test data, Eq.(4-7) is proposed to estimate the $f_{ct,sp}$ based on f_c for a curing age of 2 days to 28 days.

The static elastic modulus of the conventional precast structural concrete mixture decreases by 9% when the high-quality RCA replace the coarse fraction of limestone. The incorporation of the high-quality RCA does not influence the static elastic modulus of the SCC precast mixture. The prediction equation from Eurocode 2 [1] can accurately predict the development of the elastic modulus of the concrete in this project. There is no clear tendency that the RCA influence the static elastic modulus development. The model in ACI 209 [2] to predict the elastic modulus based on the compressive strength provides a satisfactory description of the test data in the present study, tested at the age of 2 to 28 days. Incorporating the high-quality RCA does not influence the relation between the elastic modulus and the compressive strength of precast concrete mixtures.

9.1.2 Variability of RAC

The lab tests on the conventional precast concrete showed an increase in within-batch variability of the compressive strength on a curing age of 1 day and 28 days when at least $50\%_{V/V}$ of the coarse limestone fraction is replaced by the high-quality RCA. Tests on the batch-to-batch variability showed that the mean compressive strength could vary between different batches, both for concrete with the NA or RCA. There is no clear tendency that the high-quality RCA influence the batch-to-batch variability of the compressive strength of the mixtures.

The results on the SCC precast concrete showed no statistically significant difference in within-batch variability of the compressive strength of concrete for a replacement percentage up to $20\%_{V/V}$. The incorporation of $100\%_{V/V}$ RCA in the SCC precast concrete results in increased compressive strength variability. These lab results have been validated by similar precast SCC mixtures made in an industrial precast plant. The compressive tests on 20 cubes at a curing age of 28 days showed a decrease of 2% in compressive strength when the high-quality RCA replaced the coarse limestone fraction. Incorporating $20\%_{V/V}$ of the RCA does not influence the variability of the test results. A replacement percentage of $100\%_{V/V}$ results in a statistically significant higher variability (+50%), confirming the lab tests results.

9.1.3 Bond behaviour between the RAC and steel reinforcement

From the pull-out tests is observed that the failure of the test specimens is characterised by the pull-out of the rebars from the concrete test specimen, irrespective of the RCA content, and no splitting failure is observed. The results show that the RCA do not considerably affect the maximum bond stress of the specimens; however, the normalised bond strength, defined as the ratio of the maximum bond stress to the square root of the concrete compressive strength, slightly increases with the increase of the RCA replacement percentage. The influence of the RCA on the slip at the maximum bond stress does not show a clear trend.

A new empirical model is proposed to estimate the maximum bond stress. The model is formulated as a function of the concrete compressive strength and RCA replacement percentage. The predictions of the proposed model are in good agreement with the test data in the present study as well as that in the literature. The bond-slip model in fib Model Code 2010 [39] provides a satisfactory description of the local bond-slip behaviour of the test specimens in the present study, provided that the measured maximum bond stress is used. A more accurate description of the bond-slip behaviour up to the slip s1 (=1.0 mm) can be achieved by adjusting the model's α-value.

9.1.4 Camber of prestressed RAC beams

The influence of different RCA replacement percentages on the camber of the prestressed beams is determined. The presence of the high-quality RCA results in an increased initial camber of the prestressed beams. The calculated camber is for the prestressed beam of RASCC0, RASCC20 and RASCC100, respectively 8.18, 8.41 and 8.50 mm. The results show that the measured initial camber is generally higher than the calculated initial camber, 3%, 3% and 9% for RASCC0, RASCC20 and RASCC100, respectively. The increased initial camber for the RASCC100 compared to RASCC0 is caused by the lower elastic modulus of RASCC100.

The relative evaluation of the long-term camber of the prestressed beams shows that a complete replacement of the coarse limestone fraction by the high-quality RCA results in an increased long-term camber by 20%. A replacement of up to 20%$_{V/V}$ does not affect the long-term camber development of the prestressed concrete beams. One could conclude from the results that concrete mixtures with a replacement up to 20 %$_{V/V}$ of the coarse limestone fraction do not require an additional correction factor for time-dependent properties, while a full replacement requires a correction factor of 1.20.

9.2 Future work

This work describes the first steps towards evaluating the feasibility of using RCA in precast structural concrete. The obtained results are promising, but some aspects need further attention before implementing the high-quality RCA in precast concrete.

- The batch-to-batch variability or repeatability of the concrete quality between batches has been tested in this work. No clear tendency is observed whether the high-quality RCA influence the batch-to-batch variability of the compressive strength of the mixtures. It should be noted that the tests were carried out with the high-quality RCA from the same crushing company. A more considerable variation in batch-to-batch variability is expected when different RCA sources are used. More tests on this issue are necessary to evaluate the batch-to-batch variability of the compressive strength of concrete with RCA.
- To further evaluate the feasibility of high-strength concrete with high-quality RCA in precast prestressed elements, the bond characteristics between prestressing strands and concrete with RCA need to be well understood and assessed. The presence of high-quality RCA in precast structural concrete can slightly change specific concrete characteristics, as discussed before in this thesis.
- Chapter 8 discusses the long-term deflection of unloaded prestressed beams. This type of deflection is affected mainly by the time-dependent properties of RAC. Shrinkage (drying and autogenous) and creep of concrete directly influence the prestress losses and the long-term

deflections under a sustained load. Several studies already confirmed that incorporating RCA in concrete can lead to larger long-term deformation of concrete. The measurement of shrinkage and creep of concrete is rather time-consuming and not possible in a precast plant, so prediction models can be used. Current prediction models for creep and shrinkage of concrete are based on the principles of conventional concrete with only NA.

As discussed in Chapter 8, several studies have shown that incorporating RCA in concrete affect its time-dependent properties. Previous studies have proposed a range of correction factors to estimate the shrinkage and creep of RAC [189]. However, the spread of these correction factors is high due to different RCA types. Most studies use lab-made normal-quality RCA to assess its influence on the creep and shrinkage of concrete [189]. The influence of industrially produced high-quality RCA on the creep and shrinkage of precast structural concrete with a high strength should be investigated.

- The preliminary full-scale tests discussed in Chapter 8 have been limited to measuring the camber of prestressed concrete beams. Further tests on a structural level are necessary to create a deeper understanding of the structural behaviour of RAC. As discussed in Section 5.3.4, different studies show a similar behaviour between structural concrete with RCA and natural aggregates. Some studies conclude that there is no influence of RCA on the ultimate bearing capacity of beams or that the prediction of shear strength in different codes can be applied for structural RAC [31], while other studies doubt that the predictions for shear strength made for conventional concrete can be applied to RAC. The results from limited structural tests with RAC beams are not consistent. More long-term structural tests on prestressed beams incorporating high-quality RCA with external loads are necessary to better understand the load-bearing capacity and deformation in the long term.
- The implementation of high-quality RCA in precast structural concrete should first be tested for internal applications. After successfully implementing high-quality RCA in these applications, the possibility of using high-quality RCA in exterior applications can be

assessed by testing the durability of precast structural concrete with high-quality RCA. The resistance of concrete with RCA against frost thaw and corrosion induced by carbonation and chlorides should be tested.

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