

CORNET Final Report / Schlußbericht

CRUFI

Modelling of transient thermal and moisture performance of concrete structures and prediction of reinforcement corrosion in (recycled) concrete
Modellierung des instationären Wärme- und Feuchteverhaltens und Prognose der Bewehrungskorrosion bei Beton und Recycling-Beton

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Abbreviations

CDW	Construction Demolition Waste
CRA	carbonated recycled aggregates
CRCA	coarse granulometrie
DAfStb	Deutscher Ausschuss für Stahlbeton DAfStb (German Committee for Reinforced Concrete)
DIBt	Deutsches Institut für Bautechnik (German Institute for Building Technology)
ETICS	External Thermal Insulation Composite System
FRCA	fine granulometrie
RCA	Recycled Concrete Aggregate
RC	Recycling Concrete
RH	Relative Humidity
SME	Small and Medium-sized Enterprise
WUFI	Wärme- und Feuchte instationär – (EN: heat and moisture transient) Software tool for the hygrothermal calculation of building parts

1 Executive Summary

In this project, the durability of concrete with and without chloride-containing recycled aggregate was investigated at different exposures and under consideration of the CO₂ binding capacity. Therefore, transient prediction models for reinforcement corrosion as a function of temperature, moisture, corrosivity and time will be developed.

Normally, the reinforcing steel is protected against corrosion due to the alkalinity of the concrete. When carbonation occurs, this protection can get lost. While carbonation is fast in dry concrete and much slower in a moist or wet concrete, corrosion behaves conversely. Whereas calculation models for carbonation are already available, they are missing for the transient prognosis of corrosion progress as a function of temperature, humidity, time and corrosivity of the concrete. Also detailed material properties for the time dependent liquid and vapor transport in the different areas of the concrete are required, as up to now effective properties for the transfer through the concrete layer seemed to be sufficient for the moisture resistant material.

To reduce demolition waste and limit sand and gravel mining, the use of recycled aggregates in concrete gains in importance. Therefore, also chloride-contaminated material shall be used, which hardly happens up to now due to the chloride-induced corrosion risk. However, under rather dry conditions (for example in interior walls) the use is expectable, at least when limiting the chloride concentration to an uncritical level. In addition, recycled concrete can capture high-amounts of CO₂, especially in the case of crushed aggregates of the old concrete. However, this process can additionally increase the corrosion risk. Therefore, the use of recycled concrete with chlorides and accelerated carbonation requires a clear and reliable design and definition of the application areas.

Based on this investigations hygrothermal simulation tools and evaluation models will be developed which enable the prediction for any application and operation whether and to which extent, a corrosion risk of the reinforcement has to be feared, and how this can be prevented e.g. by changing assembly or materials or by adding coatings. For that purpose, normal, recycling and CO₂ absorbing concretes are considered.

The new prediction possibilities will allow to avoid corrosion problems, develop cost efficient refurbishment measures and to extend the use of recycled concrete while improving the carbon footprint of concrete structures.

2 Introduction

The potential of recycled concrete is very high - according to German ready mixed concrete organization (BTB 2020) 26 million tons only in Germany (with an overall production of 53 million tons) and can be significantly increased if small recycling aggregates < 2 mm and especially if chloride containing aggregates from concrete roads, bridges and parking decks are allowed to be used for recycling concrete. Thus, the proven harmlessness of chloride-containing aggregates will significantly increase the applicability of recycled concrete. It can be assumed that around one third of the German producers will integrate recycled concrete into their portfolio in the medium term. The expected coverage in Europe is 20 % of the producers. In Brazil, there are about 300 private recycling plants for processing CDW. With the new national standard allowing the use of recycled concretes as aggregates in structural concretes, new market opportunities are expected for the sector, influencing also other sectors, such as the dismantling and ready mixed concrete companies.

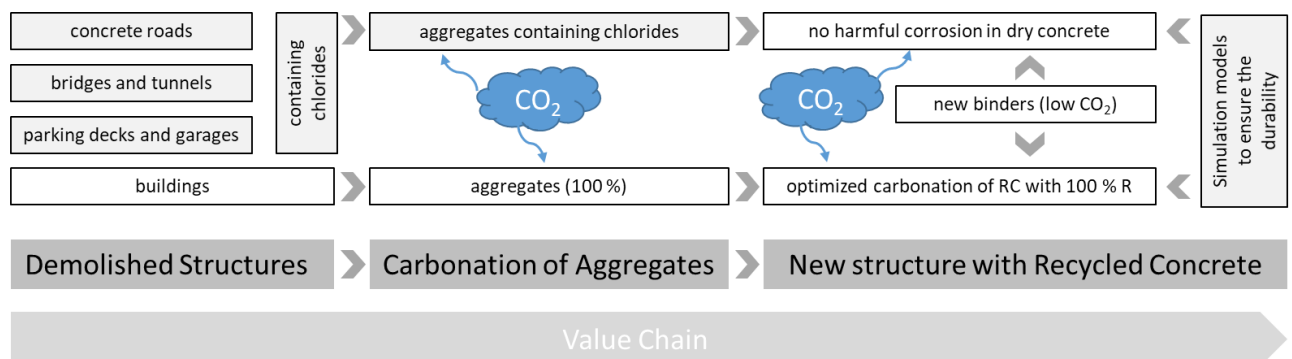


Figure 1: Schematic representation of the planned research process of the CRUFI project

The value chain (see Figure 1) of using aggregates containing chlorides starts with the demolished structure. Here the German demolishers could provide valuable input in how the aggregates are crushed and stored as part of the basic research. The carbonation of aggregates is a new link of the value chain which can either be carried out from the demolisher or the ready mixed concrete producer. The carbonation of the aggregates was characterized as well as the optimization of the carbonation process of the recycled concrete in the new structure. Also using aggregates containing chloride exposure conditions must be ensured to exclude harmful corrosion.

The following topics were covered:

- Development of Recycled Concretes with Chloride-Containing Aggregates
- Determination of the durability parameters of concretes with chloride-containing aggregates
- CO₂-integration in recycled concretes
- Development of refined hygrothermal material models for concrete, Determination of material properties and corrosion maps for the new concrete types

- Development of A corrosion Prediction model and corrosions maps for Different Concrete types
- Demonstrator Production and Big-Scale Verification of Simulation Results
- Handling instructions for RC concrete containing chloride

3 Development of Recycled Concretes with Chloride-Containing Aggregates

3.1 Overview

The objective of this study was to develop recycled concrete (RC) containing up to 100 % chloride-containing recycled aggregate (RCA). The concrete mixtures were optimised for CO₂ savings, workability, strength and durability.

Initially, the RCAs from crushed old concrete were characterised in terms of their grain size distribution, water absorption, bulk density and chloride content. The chloride content was determined both directly on the milled aggregate and indirectly in the eluate.

For exposure classes 'XC1' and 'XF1', RCs were developed with and without the addition of limestone powder. The RCs were analysed to their fresh and hardened concrete properties, as well as their durability. In particular, carbonation and frost-thaw resistance were tested.

To determine the chloride content after the concrete production, the RC samples were crushed again and the chloride content was also analysed directly on the aggregates and indirectly in the eluate.

The test procedure is shown in Figure 2.

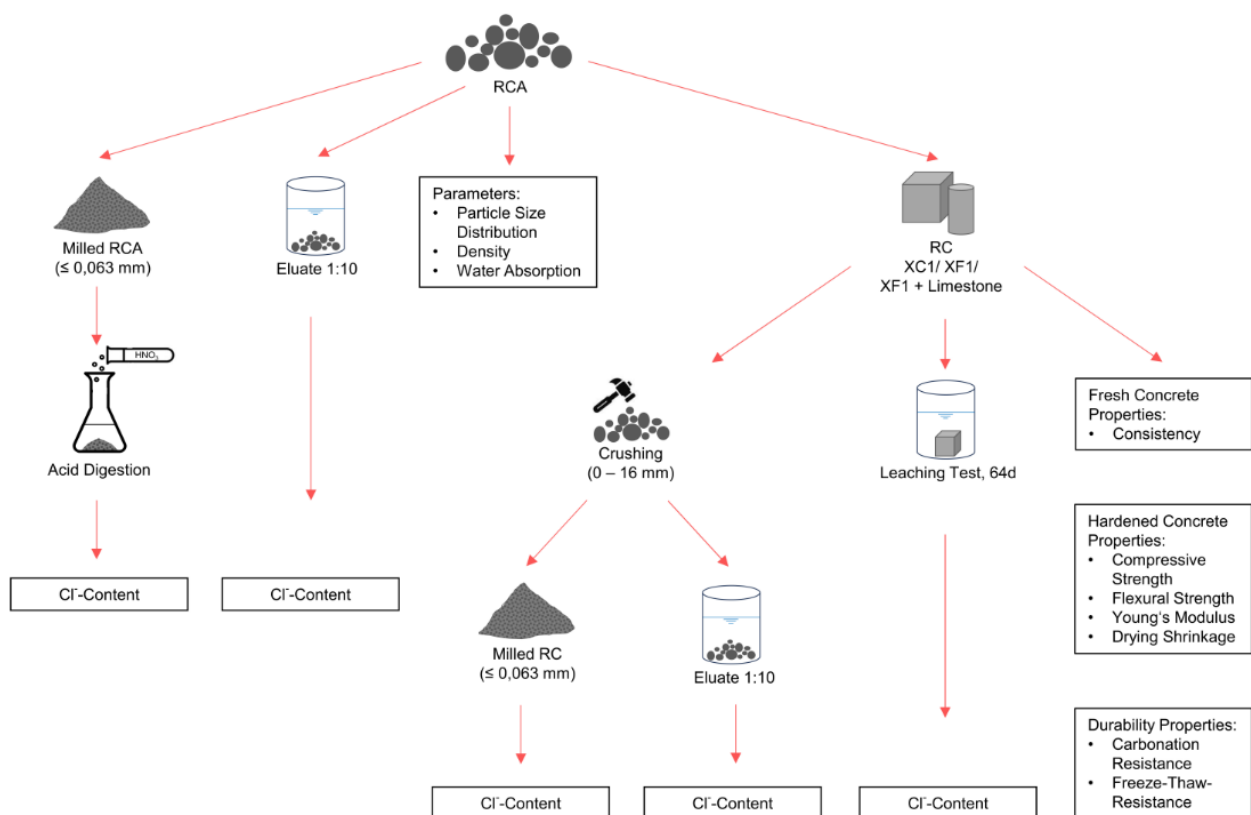


Figure 2: Schematic diagram of the RCA and TC test procedure

3.2 Identification of demolished concrete

3.2.1 Recycled concrete aggregates (RCA)

Origin

In total seven RCAs were analyzed in this study. Six of them were from different demolition construction sites, all from structures consisting of reinforced concrete. One sample was from concrete samples from the laboratory with a defined chloride content of 3.0 wt.-%/c. The used materials are listed in Table 1.

Table 1: List of used demolition materials

Demolition Material	Origin
TS	Bridge deck
CC	Bridge center cap
OC	Bridge outer cap
LS	Underground garage (complete)
GG	Underground garage (complete)
TK	Floor duplex pit of an underground garage (demand-based)
CI	Concrete laboratory samples with a predefined chloride content

Preparation

The different RCAs were crushed to a grain size distribution of 0–16 mm using a jaw crusher (see Figure 3). The RCA-TS did not require any further processing, the material was already delivered in the desired grain size fraction.



Figure 3: Demolition material (l), jaw crusher (m), RCA material with a grain size of 0–16 mm (r)

3.2.2 Characterisation of the RCAs

Particle size distribution

The grading curve was determined for each crushed RCA in accordance with the (DIN EN 933-1 2012). The grading curves of the RCAs are mostly in the range between A16 and B16 (see Figure 4), also the grading curve of the RCA of the used reference mixture. For the RCAs GG, TK, CI and the Reference without chlorides the prepared material was sieved into 0/4, 4/8 and 8/16 mm fractions, from which the standard A/B16 grading curve was created. The other RCAs (TS, CC, OC, LS) were not sieved separately into fractions; they were used after crushing.

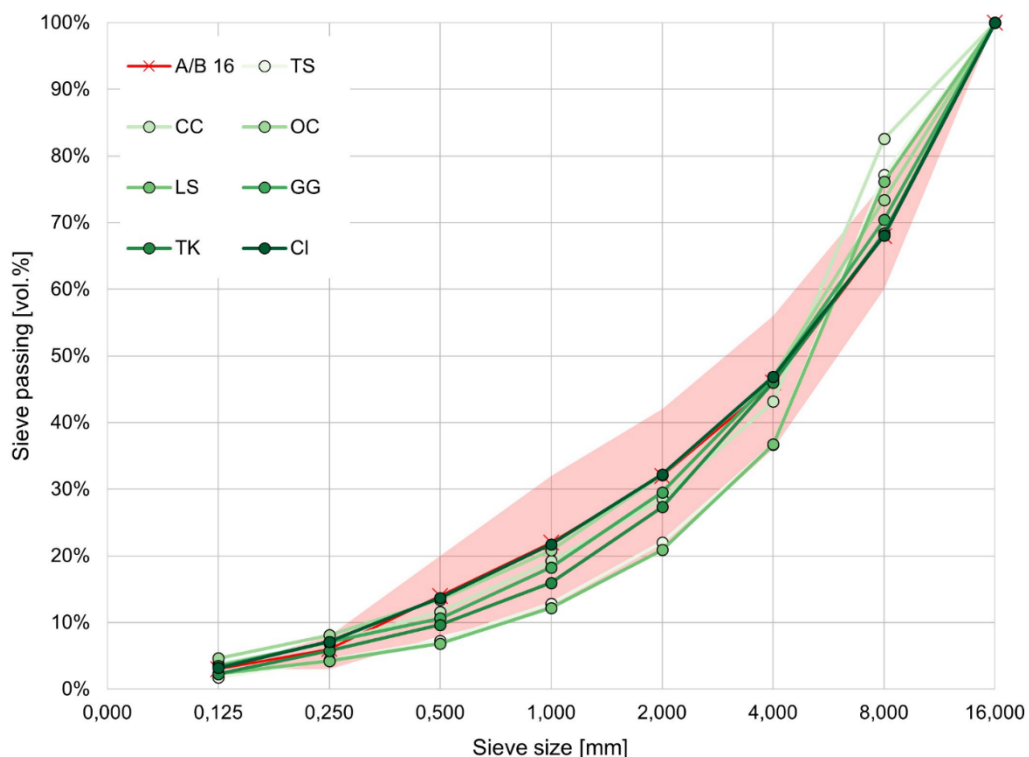


Figure 4: Particle size distribution of the RCAs, range of the standard sieve line A/B16 (red)

Density and water absorption

The water absorption and bulk density of the RCAs were determined over the entire A/B16 sieve curve. The tests were carried out according to (DIN EN 1097-6 2022). The bulk density was measured using a glass pycnometer. For the water absorption, an immersion and pumping time of 5 minutes each was chosen. The results are summarized in Table 2.

Table 2: Density and water absorption of the RCAs

RCA	Density ρ [kg/dm ³]	Water Absorption WA_5 [%]
TS	2.40	6
CC	2.37	6
OC	2.38	6
LS	2.36	7
GG	2.36	7
TK	2.42	6
CI	2.33	7

3.3 Sample preparation

3.3.1 Recycled concrete (RC)

Exposure classes 'XC1' for internal components and 'XF1' for external components according to (DIN EN 206 2021) were selected for the production of the concrete samples. The concrete samples were made from 100 % RCA, including the sand fraction (≤ 2 mm). In addition, samples containing 20 % limestone powder were produced (marked with the abbreviation 'L'). To pre wet the RCAs, 80 % of the measured water absorption was selected. The mixtures with the RCAs GG, TK and CI are optimized mixtures, based on the results of previous test series. In order to evaluate the durability tests, a chloride-free reference mixture was produced so that frost-thaw resistance could be evaluated without the influence of chloride. The composition of the concrete mixtures is shown in Table 3.

Table 3: Composition of concrete mixtures [kg/m³]

	TS	CC	OC	LS	GG	GG-L	TK	CI-L	Ref.
Exposure class	XC1	XC1	XC1	XC1	XF1	XF1	XC1	XF1	XF1
CEM I 42.5 N	300	300	300	300	400	320	360	320	320
Water	165	165	165	165	200	190	198	190	190
(w/b)	(0.55)	(0.55)	(0.55)	(0.55)	(0.50)	(0.48)	(0.55)	(0.48)	(0.48)
Limestone	-	-	-	-	-	80	-	80	80
Extra water to pre wet (80 % of WA ₅)	83	82	83	96	87	88	78	86	90
RCA	1736	1714	1721	1707	1548	1563	1624	1543	1616

3.3.2 Eluate

The eluates of the RCAs were prepared in accordance to the standard (DIN EN 1744-3 2002). For this purpose, the RCAs and the RC-samples were stored as crushed material with a grain size distribution of 0-16 mm for 24 h in an overhead shaker with deionised water and then filtered, see Figure 5.



Figure 5: RCA in overhead shaker for 24 h (l), filtered Eluate (r)

3.4 Testing methods

3.4.1 Fresh Concrete

Consistency

The consistency of the fresh RC containing the chloride containing aggregates was determined by a slump test in accordance with (DIN EN 12350-5 2019). The aim was to achieve a consistency between 'plastic' (F2) and 'soft' (F3) to ensure good workability.

3.4.2 Hardened Concrete

Compressive and flexural strength

The compressive and flexural strengths of the RC-samples were tested after 28 days, in accordance with (DIN EN 12350-3 2019) and (DIN EN 12390-5 2019). Additionally, the compressive strength was tested again after 125 days.

Young's Modulus

The secant modulus of elasticity, Young's Modulus in compression was determined in accordance with (DIN EN 12390-13 2021).

Drying shrinkage

The drying shrinkage was determined in accordance with (DIN EN 12390-16 2019) on prisms of the CI-L samples. The total shrinkage was determined by measuring the length.

3.4.3 Chloride Content

The chloride content was determined on the RCA and the RC samples. The chloride content was measured directly on the milled RCA (particle size ≤ 0.063 mm) and indirectly on the eluates. The milled samples were dissolved in acid in accordance with (DIN EN 14629 2007). The chloride contents of the eluates were determined directly on the eluate. The chlorides were measured in wt.-% using ionchromotography (IC) according to (DIN EN ISO 10304-1 2009).

In addition, the leaching behavior of the RC samples was investigated. The leaching test was performed in accordance with (DIN CEN/TS 16637-2 2014). The RC samples were eluted at different time intervals over a period of 64 days, after which the chloride content of the eluate was analyzed using IC.

3.4.4 Durability

Carbonation resistance

The carbonation resistance was tested on the GG and GG-L samples in accordance with (DIN EN 12390-10 2018). The tests were carried out both using an accelerated procedure in a climate chamber and under natural storage conditions in atmospheric conditions.

Freeze-Thaw resistance (CIF)

The freeze-thaw resistance was determined on the CI-L-samples in accordance with the BAW Frost Test of Concrete (BAW 2012). These samples were selected because they had the highest chloride content of all RCAs. It was expected that if the RC mixtures with the highest chloride content passed the durability tests, then all other RCAs with chloride-containing aggregates would also pass. For verification purposes, additional reference samples were tested which were produced using the same mix design as the CI-L samples - also with RCA, but without chlorides.

3.5 Experimental results and discussion

3.5.1 Fresh Concrete

Consistency

The consistency of the fresh RC produced with chloride containing aggregates ranged from plastic (F2) to soft (F3), shown in Table 4. No clear correlation could be identified between the w/b-ratio and consistency. This is probably due to the use of RCA, which influences the behavior of fresh concrete due to its variable water absorption.

Table 4: Consistency of the RC-samples

	TS	CC	OC	LS	GG	GG-L	TK	CI-L
Slump flow [mm]	438	388	412	412	413	345	443	388
Consistency	F3 soft	F2 plastic	F2/F3 soft/ plastic	F2/F3 soft/ plastic	F2/F3 soft/ plastic	F2 plastic	F3 soft	F2 plastic
w/b	0.55	0.55	0.55	0.55	0.50	0.48	0.55	0.48

3.5.2 Hardened Concrete

Compressive and flexural strength

The results of the strength tests on the RC samples are shown in Table 5. The compressive strength tests were carried out after 28 d and additionally after 125 d. The flexural strength tests were carried out after 56 d. The value in parentheses indicates the standard deviation of the corresponding strength.

Table 5: Compressive strength [N/mm²] after 28 and 125 days, flexural strength [N/mm²] after 56 days

	TS	CC	OC	LS	GG	GG-L	TK	CI-L
$f_{cm, cube (28d)}$ [N/mm ²]	23.2 (0.5)	-	29.5 (-)	23.3 (1.3)	36.5 (2.2)	38.2 (0.9)	30.3 (1.0)	35.1 (0.8)
$f_{cm, cube (125d)}$ [N/mm ²]	-	-	-	-	37.7 (0.0)	38.0 (0.9)	31.6 (1.6)	35.4 (3.7)
$f_{ct, fl (56d)}$ [N/mm ²]	-	-	-	-	-	-	-	3.5 (0.2)

Due to the small quantity of the RCAs, it was not possible to determine the tests for all RC-samples. It should be noted that only one cube was tested for the OC-sample ($n = 1$). For the compressive strength after 128 d, only two cubes ($n = 2$) were tested, while a series of $n = 3$ were tested for the other samples.

The RC-samples with optimized concrete mixtures (GG, GG-L, TK, CI-L) generally achieved higher strength values. With a compressive strength of 23.2 N/mm², the TK-sample corresponded to the compressive strength class C16/20. This fulfills the minimum compressive strength required for the target exposure class 'XC1'. The GG-L mixture, in which 20 % of the cement was replaced by limestone powder, achieved the highest compressive strength of 38.2 N/mm². The GG, GG-L and CI-L mixtures produced for exposure class 'XF1' also fulfilled the requirements for compressive strength C25/30.

The flexural strength was tested on test specimens from the shrinkage tests for the RC sample CI-L. After 56 days, this sample achieved a flexural strength of 3.5 N/mm².

Young's Modulus

The Young's Modulus was determined after 28 days. The results are shown in Table 6. The values in parentheses indicates the standard deviation.

Table 6: Young's Modulus [N/mm²] after 28 days

	TS	CC	OC	LS	GG	GG-L	TK	CI-L
Young's Modulus [N/mm ²]	9.500 (-)	-	-	9.400 (100)	20.900 (400)	19.900 (1100)	24.200 (800)	-

Due to the limited amount of the RCAs, not all tests could be carried out for all RC-samples. It should be noted that only one specimen ($n = 1$) was tested for the TS-sample, while two specimens ($n = 2$) were tested for the LS-sample. Three or more samples were tested for the others.

The RC-samples with the optimized concrete mixtures (GG, GG-L, TK) achieved higher young's modulus than the samples TS, LS. The highest modulus of elasticity of 24.200 N/mm² was measured in the TK-sample.

Drying shrinkage

The results of the shrinkage test for the CI-L samples are shown in Figure 6, with length measurements taken on days 6, 14, 27 and 56.

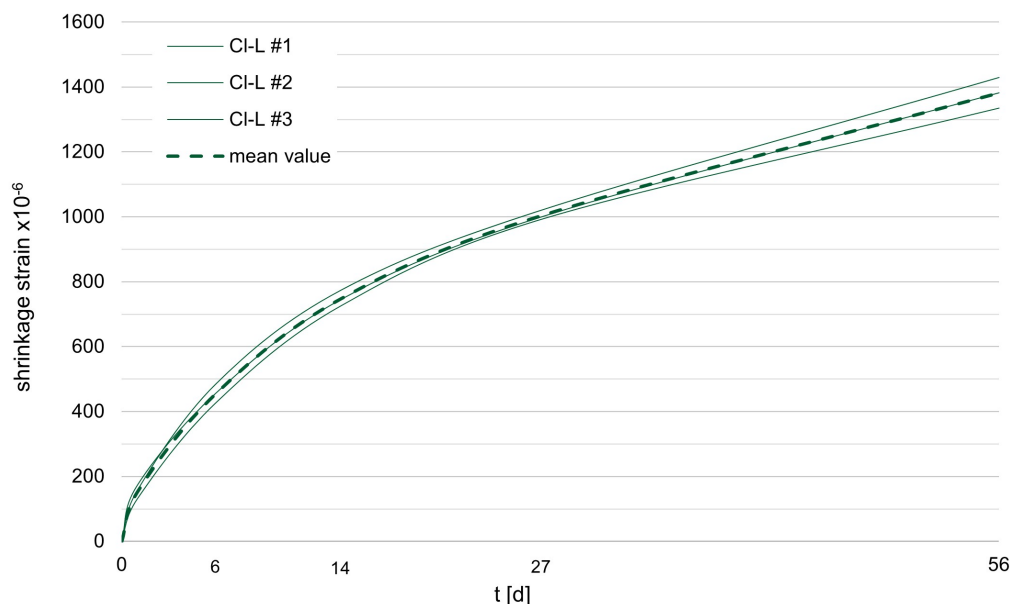


Figure 6: Shrinkage strain $\times 10^{-6}$ from the CI-L-samples

All three test specimens showed similar values at the respective measurement times, as well as identical curve progression. The strongest increase in shrinkage occurred within the first 14 days and did not really approach to a constant end value until the age of 56 days.

Overall, the shrinkage strain was comparatively high at approximately 1.400×10^{-6} . This is probably due to the use of 100 % RCA (including the sand fraction), which has a higher water absorption capacity and thus significantly affects shrinkage behavior.

3.5.3 Chloride Content

The results of the chloride contents of the RCAs and the RCs are shown in Figure 7. The measured values are compared with the German standard limits: the normative limit in the eluate according to (0.015 wt.-%) (DIN 4226-101 2017) and the maximum corrosion-inducing chloride content (0.06 wt.-%) according to (DIN EN 1045-2 2023).

It should be noted that not all chloride values could be determined for the RC-samples due to the partially limited quantities of RCA, as indicated for the samples CC, LS.

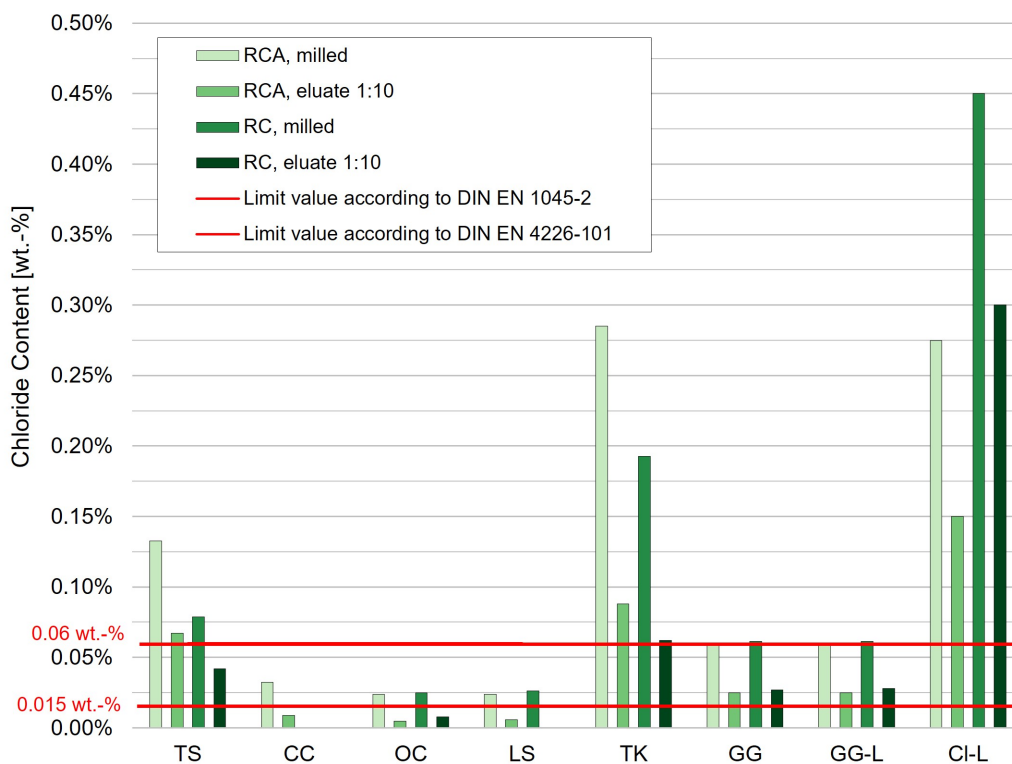


Figure 7: Chloride content [wt.-%] of the RCA and the RC milled and in the eluate compared with the German limit values

As expected, the milled samples show higher chloride concentrations than the eluates. This is due to the fact that the milled samples were dissolved in acid, which also releases bound chlorides.

The chloride contents of the RCA samples show clear differences between the different RCAs. The aggregate CC, OC, LS, GG is below both limits in both the eluate and the milled material. In contrast, samples TS, TK and CI clearly exceed the limit values. The chloride content of RCA CI is 0.28 wt.-%, which corresponds to a value of 1.93 wt.-%/c in relation to the cement content. This is below the originally defined chloride content of 3.0 wt.-%/c, which was specifically introduced during the production of the CI-L laboratory samples.

When comparing the chloride content of the RC-samples with the corresponding RCA-samples, it is noticeable that the chloride concentration in the RC is higher in CI-L than in the RCA. This finding is unusual as the chloride content is usually reduced by the addition of

binder during concrete production. In this case, it must be considered that CI-L is not old concrete, they were laboratory samples specially prepared after 28 days and then reprocessed. In the case of the other RCAs, the chlorides gradually leached into the structure from the surface of the concrete over a period of years, whereas in the case of the CI-L RCA, the chlorides were specifically added to the mixing water during the production of the concrete. The different method of chloride introduction could significantly influence the binding behavior of the chlorides and therefore explain the increasing chloride content in the concrete samples. On the other hand, the RC samples of OC, LS, GG and GG-L do not show any significant deviation from the starting material, which could be due to the already very low chloride content and the associated measurement inaccuracy. Sample TS and TK shows an expected decrease in chloride content of about 35 %, which corresponds well with the ratio of about 35 % binder to 65 % aggregate in the RC, see Table 3.

Figure 8 shows the evolution of chloride release based on a leaching test over 64 days. The chloride concentrations in the eluates at different test times are shown. This test was carried out on the optimized concrete formulations for RCAs TK, GG and CI-L.

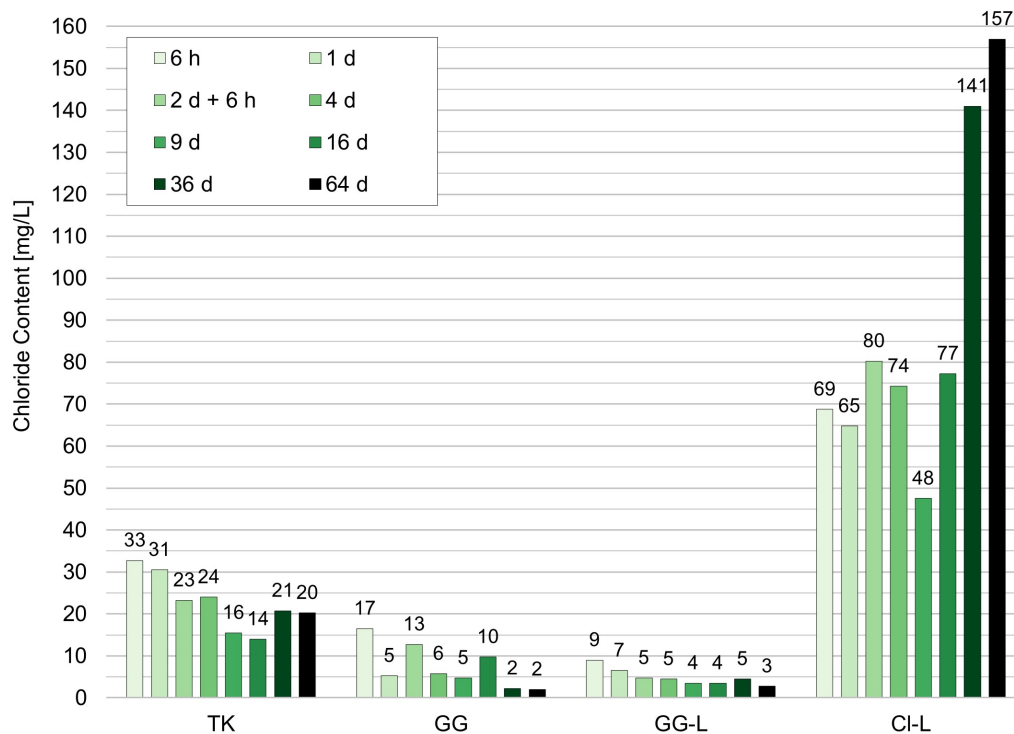


Figure 8: Chloride content [mg/L] of the leaching test over 64 d

The basic picture corresponds to the previous observations: Samples GG and GG-L have the lowest chloride contents, while CI-L consistently shows the highest values. In samples TK, GG and GG-L the measured chloride concentration decreases with increasing leaching time. This behavior indicates that the dissolved chlorides are leached out early and the remaining parts are bound.

The CI-L-samples show a different behavior. Here the chloride content does not decrease with time, but increases significantly after 36 and 64 days. This behavior indicates a high proportion of unbound chloride, which is released by diffusion during the test. A possible reason for this is the origin of the CI-L samples. They are from laboratory tests in which a defined amount of chloride was added to the concrete mix. As a result, the binding behavior of the chlorides may be different from that of the RCA, TK and GG samples. The chlorides may therefore be present in free form in the CI-L-samples.

3.5.4 Durability

Carbonation resistance

Figure 9 and Table 7 show the results of the carbonation resistance test. The test specimens were stored under the following conditions:

- Atmospheric storage: 20 °C, natural CO₂ concentration, for 273 days.
- Climate chamber: 20 °C, 65 % relative humidity and 3 % CO₂; after 273 days of atmospheric storage, the specimens were placed in the climate chamber for an additional six days.

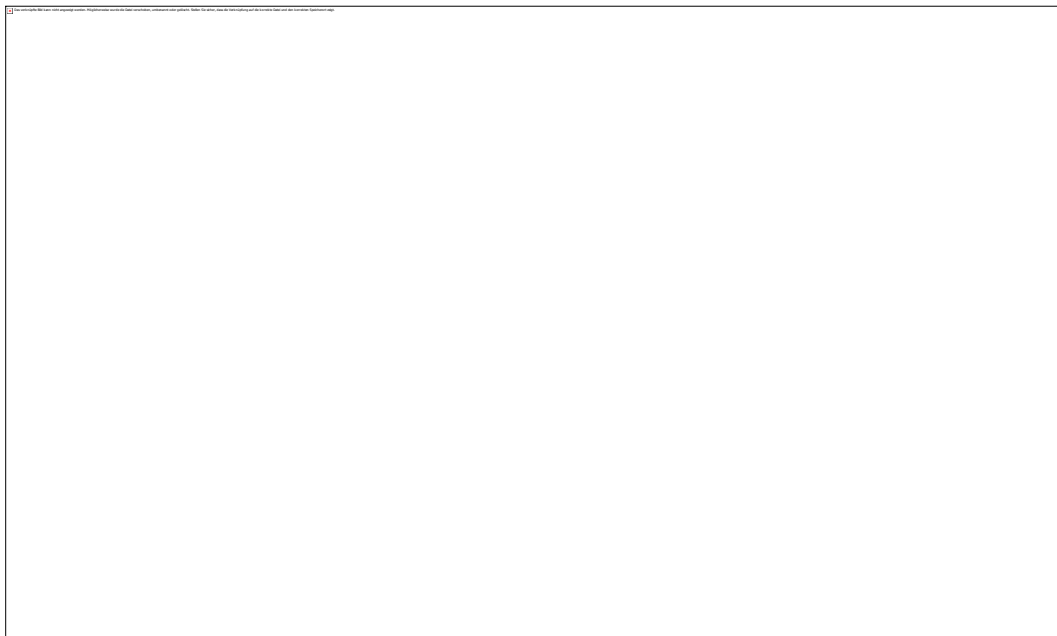


Figure 9: Carbonation depth [mm] of the RC-samples GG and GG-L as a function of exposure time [$\sqrt{t}$]

RC GG-L, which was produced using limestone powder, exhibits a slightly higher carbonation rate than RC GG without limestone powder, under both accelerated conditions in the climate chamber and during atmospheric storage. This may be because part of the cement in RC GG-L was replaced by limestone powder, resulting in less calcium hydroxide being available. As expected, carbonation under atmospheric conditions is significantly slower than under accelerated laboratory conditions.

Table 7: Carbonation rate [mm/ $\sqrt{t}$]

RC	Carbonation rate – 273 d atmospheric k_{ac} [mm/ $\sqrt{t}$]	Carbonation rate – 273 d atmospheric, 6 d climate chamber k_{ac} [mm/ $\sqrt{t}$]
GG	0.19	3.09
GG-L	0.26	3.34

The carbonation rates determined for atmospheric storage (0.19 mm/ $\sqrt{t}$ for GG and 0.26 mm/ $\sqrt{t}$ for GG-L) indicate a high level of carbonation resistance.

Freeze-Thaw-Resistance

The results of the CIF test of the CI-L-samples are shown below in comparison with a reference. Figure 10 shows the mass loss due to weathering during 56 freeze-thaw cycles. According to the BAW, the test is passed if the mass loss after 28 freeze-thaw cycles is less than 1000 g/m² (BAW 2012).

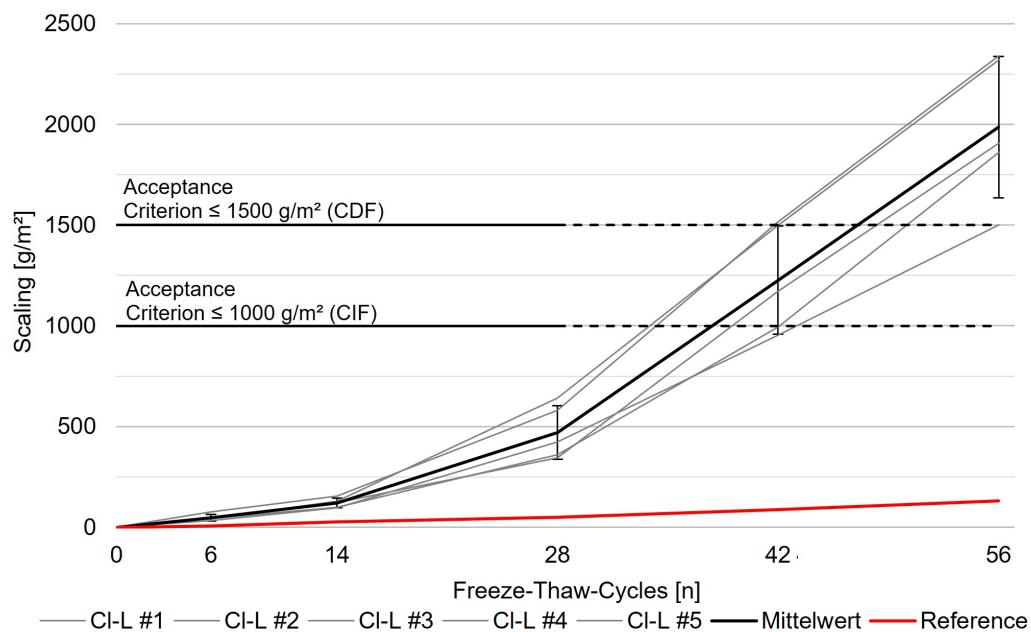


Figure 10: Scaling [g/m²] of the CIF-tests over 56 d RC-samples CI-L and a reference

The results show that the CI-L samples comply this requirement and therefore successfully pass the suitability test. However, it is noticeable that the reference samples - consisting of 100 % RCA with no chloride content - show significantly lower scaling rates. A possible explanation for this is the chloride contamination of the CI-L samples. It is possible that free chlorides are released from the RC structure during the freeze-thaw cycles, resulting in an increased chloride concentration in the test fluid itself, see also Figure 8. This could cause

additional damage to the concrete surface and explain the higher weathering of the CI-L samples observed.

Figure 11 shows the visual changes to the test surface after 0, 28 and 56 freeze-thaw-cycles. Progressive structural damage to the concrete surface can be seen as the number of cycles increases. Prior to the start of the test, the surface had a smooth structure with fine hairline cracks. After just 28 cycles, progressive weathering of the cement paste was observed. In some cases spalling occurred, locally exposing the coarse aggregate. After 56 freeze-thaw cycles, the top layer of cement paste was almost completely worn away.



Figure 11: Visual changes in the surface of the CIF test specimens CI-L after 0 cycles (l), 28 cycles (m) and 56 freeze-thaw cycles (r)

3.6 Identification of demolished concrete

3.6.1 Recycled concrete aggregates studied in Brazil

The recycled aggregates were produced by crushing first-group PAVI-S type pavers, in accordance with Brazilian Standard (ABNT NBR 9781 2013). These pavers, with an average compressive strength of 46.3 MPa, had been in use for at least 15 years. They were extracted during maintenance work and transported to a recycling plant, where they were crushed into both fine (FRCA) and coarse (CRCA) granulometries. The materials used in this research are shown in Figure 12.



Figure 12: Materials used for research. Left: Concrete pavers; center: coarse recycled aggregate; right: fine recycled aggregate.

The average density and water absorption of the pavers were 2.19 g/cm³ and 3.80 %, respectively. The pH analysis showed that, although some carbonation occurred at the edges of the pavers, there remained an alkaline content within the samples, as the conventional colour changed to purple after spraying the phenolphthalein solution (see Figure 13). This suggests that natural exposition was insufficient to fully carbonate the pavers, even after at least 15 years of exposure to atmospheric CO₂.



Figure 13: Paver's surface showing that carbonation is restricted to its borders

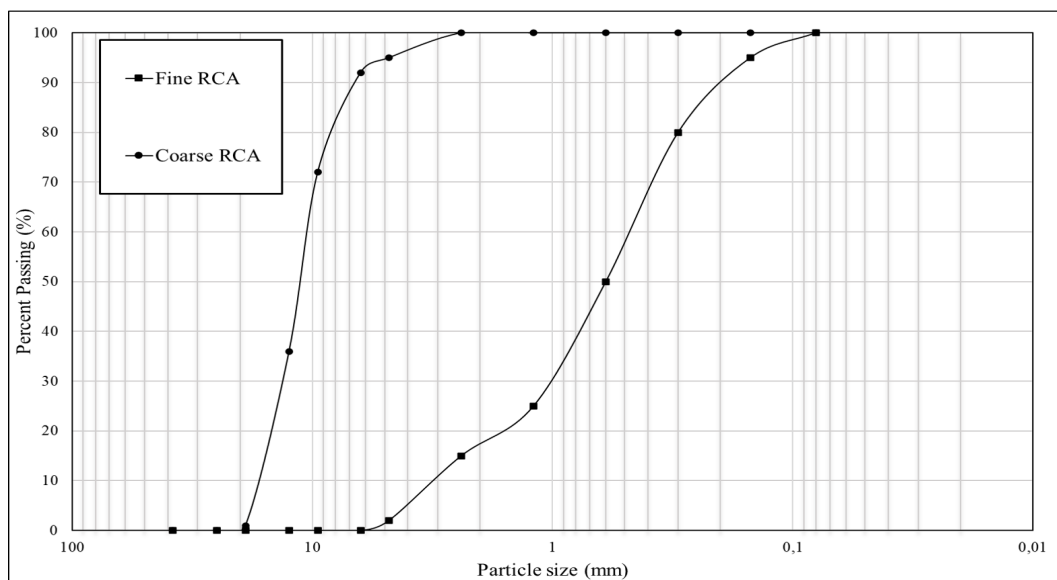
As chloride attack due to de-icing processes is not a concrete durability concern in Brazil, the recycled aggregates studied were not contaminated by Cl⁻. One of the main objectives of the study in Brazil was to investigate the carbon capture potential of the recycled aggregates. This occurs due to the carbonation of the mortar adhered to the surface of the recycled ag-

gregates. Then, for the fine and coarse aggregates obtained from the concrete pavers recycling, the amount of mortar attached to the aggregates surface was treated by acid attack (see Figure 14). It was found that the amount of mortar adhered to recycled aggregates were 28.2 % (weight basis) and 31 % for gravel and sand, respectively.



Figure 14: Coarse recycled aggregates before and after the acid attack

The granulometric distribution of both fine and coarse recycled aggregates is shown in Figure 15. According to the Brazilian standard NBR 7211, the fine fraction ranges from 0.15 to 4.75 mm, while the coarse fraction ranges from 4.75 to 20 mm. Both fine and coarse recycled concrete aggregates (RCA) exhibited water absorption of 7.32 % for fine RCA and 5.98 % for coarse RCA. The density of the RCA is 2.22 g/cm³ and 2.37 g/cm³ for coarse and fine fractions, respectively.



3.6.2 Recycled concrete aggregates studied in Austria

The concrete recycling material used for classification and sampling was obtained from various sources and specifically selected with different material qualities in order to be able to represent as broad a test spectrum as possible.

A total of four different types of recycled material from recycling companies for mineral wastes were examined, whereby in all cases a material mixture from different sources was involved. The material was classified in the course of the crushing and preparation process according to the existing impurities and contents of foreign substances in the material mixture. Since recycled aggregates from recycling plants or processing plants with intermediate storage are crushed concrete from different buildings of different ages and environmental exposure with variable concrete types and compositions, there is no uniform degree of carbonation of the crushed aggregate. It should be noted that although the majority of the recyclate is carbonated on the surface due to storage under free weathering, the interior of the grain could be assessed as still capable of absorbing CO₂ using the phenolphthalein test (see Figure 16).

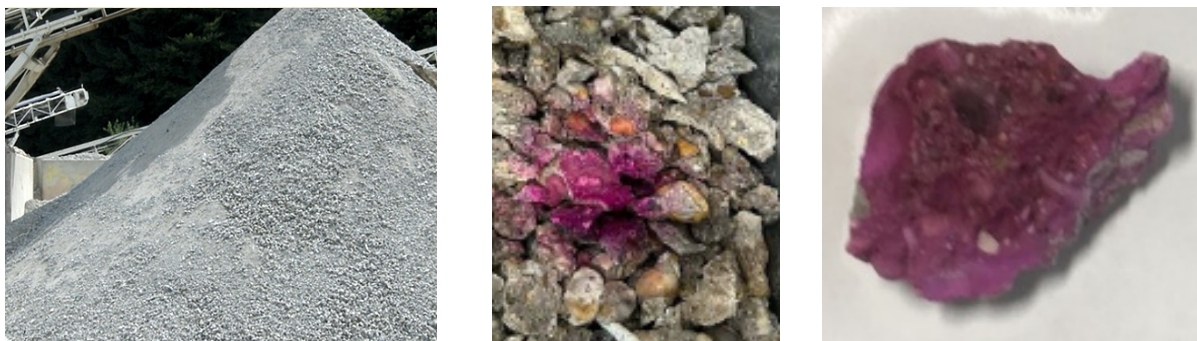


Figure 16: Concrete recyclate during preparation and storage (left), surface phenolphthalein test (center) and on broken aggregate (right)

Accordingly, the suitability of concrete rubble for carbon dioxide storage can in principle be determined via the carbonation of the cement paste after the building has been dismantled. Since a qualitative test by determining the pH value cannot provide precise information on the amount of CO₂ that can be absorbed or stored, quantitative analyses are carried out in WP 3, which also provide the influences of various reaction parameters in an accelerated process.

Quality classes and origin of the recycled aggregates investigated:

- Single-variety crushed concrete RB-A1, 0/32
 - o Pure crushed concrete from a known source and single-variety processing with strength class C25/30, exposure class XC 3
- Concrete recyclate RB II, 0/63, U7, U-A

- o Processed material from a building materials recycling plant; recycled, crushed concrete granulate with ≥ 90 M-% concrete content, aggregate for unbound and hydraulically or bituminously bound use
- Concrete recycle RB III, 0/32, U-A
 - o Processed material from a building materials recycling plant; recycled, crushed concrete granulate with ≥ 90 M-% concrete content and ≤ 10 M-% masonry bricks, aggregate for unbound and hydraulically or bituminously bound use
- Building construction recycle RMH III 0/63, U-A
 - o Processed material from a building materials recycling plant; additional tests in accordance with the application-specific regulations (ÖNORM, RVS, ...) are required for proof of suitability.

Table 8: Excerpt of the (Recycling-Baustoffverordnung, BGBl. II Nr. 181/2015) current version of the “Recycling-Baustoffverordnung” (Recycled Building Materials Ordinance), overview of permissible areas of application and quality classes of recycled aggregates

Anhang 4

Zulässige Einsatzbereiche und Verwendungsverbote gemäß den §§ 13 und 17

Tabelle 1: Tabellarische Zuordnung der Qualitätsklassen zu den Einsatzbereichen und Verwendungsverböten gemäß den §§ 13 und 17

Qualitätsklasse	Beschreibung	Ungebundene Anwendung ¹⁾ ohne gering durchlässige, gebundene Deck- oder Tragschicht	Ungebundene Anwendung ¹⁾ unter gering durchlässiger, gebundener Deck- oder Tragschicht	Herstellung von Beton ab der Festigkeitsklasse C 12/15 oder der Festigkeitsklasse C 8/10 ab der Expositions-kategorie XC1	Herstellung von Asphaltmischgut
U-A (ungebunden – A)	Gesteinskörnungen für den ungebundenen sowie für den hydraulisch oder bituminös gebundenen Einsatz	Ja	Ja	Ja	Ja
U-B (ungebunden – B)	Gesteinskörnungen für den ungebundenen sowie für den hydraulisch oder bituminös gebundenen Einsatz	Nein	Ja ²⁾	Ja	Ja

Table 9: Overview of the suitability and possible uses of recycled building materials

Tabelle 2: Anwendung und bautechnischer Einsatz von Recycling-Baustoffen

		Recycling-Baustoff		RA				RB				RAB				RM / RG				RMH		RS	RZ	RHZ	RH	Qualitätsklassen								
		Güteklasse		I	II	III	IV	SJ	II	III	IV	I	II	III	IV	SJ	II	III	IV	III	IV	III	III	III	III	U-A ¹⁾	U-B ¹⁾	U-E ¹⁾	H-B	B-B	B-C	B-D	D	
EN 12424	ungebundene und hydraulisch gebundene Gemische	obere Tragschicht	gebunden	RVS 08.17.01	(✓)	(✓)	(✓)	✓	(✓)	(✓)		✓	(✓)	(✓)		✓	(✓)	(✓)								✓	(✓) ¹⁾	(✓) ²⁾						
		obere Tragschicht	ungebunden	RVS 08.15.01	✓ ¹⁾	(✓) ²⁾		✓	(✓)			✓	(✓)			✓ ²⁾	(✓)									✓	(✓) ¹⁾	(✓) ²⁾		(✓) ³⁾		(✓) ¹⁰⁰⁰		
		untere Tragschicht		RVS 08.15.01	✓ ²⁾	✓ ²⁾⁽³⁾		✓	✓			✓	✓			✓ ²⁾	(✓) ²⁾									✓	(✓) ¹⁾	(✓) ²⁾						
		Untergrundverbesserung / Stabilisierung		RVS 08.03.01	(✓)	(✓)	(✓)	(✓)	✓	✓	✓	(✓)	✓	✓	(✓)	✓	✓	✓	✓	(✓)	+			+	+	+	✓	(✓) ¹⁾						
		Schüttungen		RVS 08.03.01	(✓)	(✓)	(✓)	(✓)	✓	✓	✓	(✓)	✓	✓	(✓)	✓	✓	✓	✓	(✓)	✓	(✓)		(✓)	(✓)	(✓)	✓	(✓) ¹⁾						
		Verfüllungen von Leitungsgräben und Überschütten von Leitungen		RVS 08.03.01	(✓)	(✓)	(✓)	(✓)	✓	✓	✓	(✓)	✓	✓	(✓)	✓	✓	✓	✓	(✓)	✓	(✓)		(✓)	(✓)	(✓)	✓	(✓) ¹⁾						
		Bettungsmaterial für Leitungszonen		RVS 08.03.01	(✓)	(✓)	(✓)	(✓)	✓	✓	✓	(✓)	✓	✓	(✓)	✓	✓	✓	✓	(✓)	(✓)		✓	(✓)	(✓)	(✓)	✓	(✓) ¹⁾						
		Hinterfüllungen und Überschüttungen von Bauwerken		RVS 08.03.01	(✓)	(✓)	(✓)	(✓)	✓	✓	✓	(✓)	✓	✓	(✓)	✓	✓	✓	✓	(✓)	✓	(✓)					✓	(✓) ¹⁾						
		fließfähiges selbstverdichtendes Künnetfüllmaterial		ONR 23131																		+	+	+	+	+	+	✓	(✓) ¹⁾					
		Drainagematerial (ausgenommen Deponiebau)																				✓	(✓)				✓	(✓) ¹⁾						
Recycling-Baustoffe im Deponiebau ¹⁾			✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	(✓) ¹⁾							
Sportstättenbau als Deckschicht und Streuschicht																				✓ ¹⁾			✓ ¹⁾	(✓) ¹⁾	✓									
Substrate zur Begrünung																				✓ ¹⁾			✓	✓ ¹⁾	(✓) ¹⁾	✓								

		Recycling-Baustoff		RB-A1				RB-A2				RB-A3				RMH-C		RH-B				Qualitätsklassen							
																						U-A ¹⁾	U-B ¹⁾	U-E ¹⁾	H-B				
EN 12620	Beton	bis C 12/15 bzw. C 8/10 unter XC1						(✓)								(✓)		+		(✓)		✓	(✓) ¹⁾	(✓) ²⁾					
		ab C 12/15 bzw. C 8/10 über XC1						(✓)										+		(✓)		✓	✓	✓	✓				
		Mauer-, Hohlblocksteine																		✓		✓	(✓) ¹⁾						
		Leichtbeton																+		(✓)		✓	(✓) ¹⁾						
		Estrich und Estrichschüttungen																+		(✓)		✓	(✓) ¹⁾						

		Recycling-Baustoff																						Qualitätsklassen									
																								U-A ¹⁾	U-B ¹⁾	U-E ¹⁾	Asphaltemischgut						
																												B-B aus B, B, B, C	B-D aus B, B, B, C	D aus B, B, B, C			
EN 13108	Asphalt	bituminös gebundene Deckschichten (Asphaltschichten)		RVS 08.16.01 RVS 08.16.06																				✓				✓	✓		✓	✓ ¹⁾⁽²⁾	✓ ¹⁾⁽³⁾
		bituminös gebundene Tragschichten (Asphaltschichten)		RVS 08.16.01 RVS 08.16.06																				✓				✓	✓		✓	✓ ¹⁾⁽²⁾	✓ ¹⁾⁽³⁾

Due to the relatively high inhomogeneity of the sample material in the case of only a small sample quantity, a relatively large sample quantity per sample type was used as the basis for the material determination and analysis, regardless of the material type. Recycled materials that are not primarily intended for use in concrete production as a substitute for natural aggregates were also deliberately selected for sampling.



Figure 17: Example of the samples received (illustrated RB II 0/63, U7, U-A) of the processed recycled aggregates

Grading curves of the recycle samples

- Single-variety crushed concrete RB-A1, 0/32

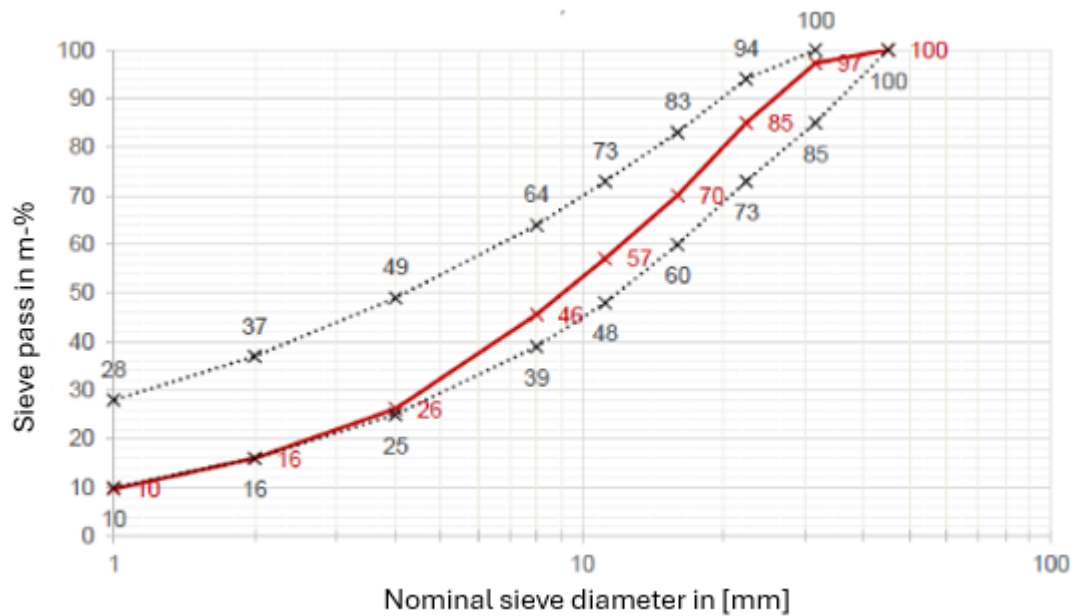


Figure 18: Test-determined grading curve of single-variety recycled concrete aggregate 0/32

- Natural aggregate, size 0/16

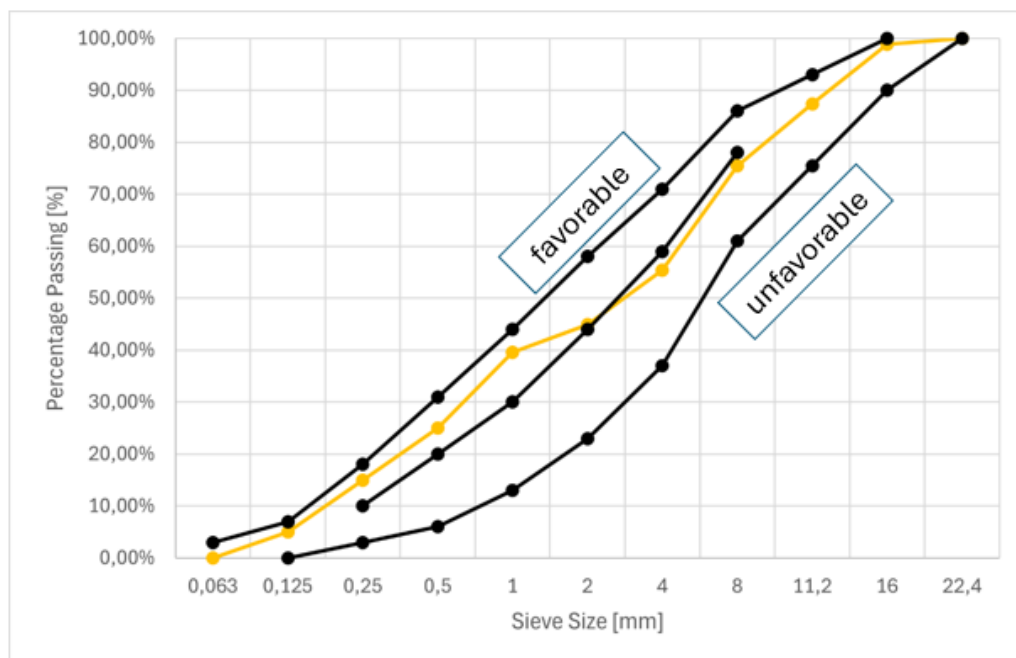


Figure 19: Grading curve of the natural aggregate used for sample production according to Austrian Standards 0-16mm

- Concrete recycle RB II, 0/63, U7, U-A

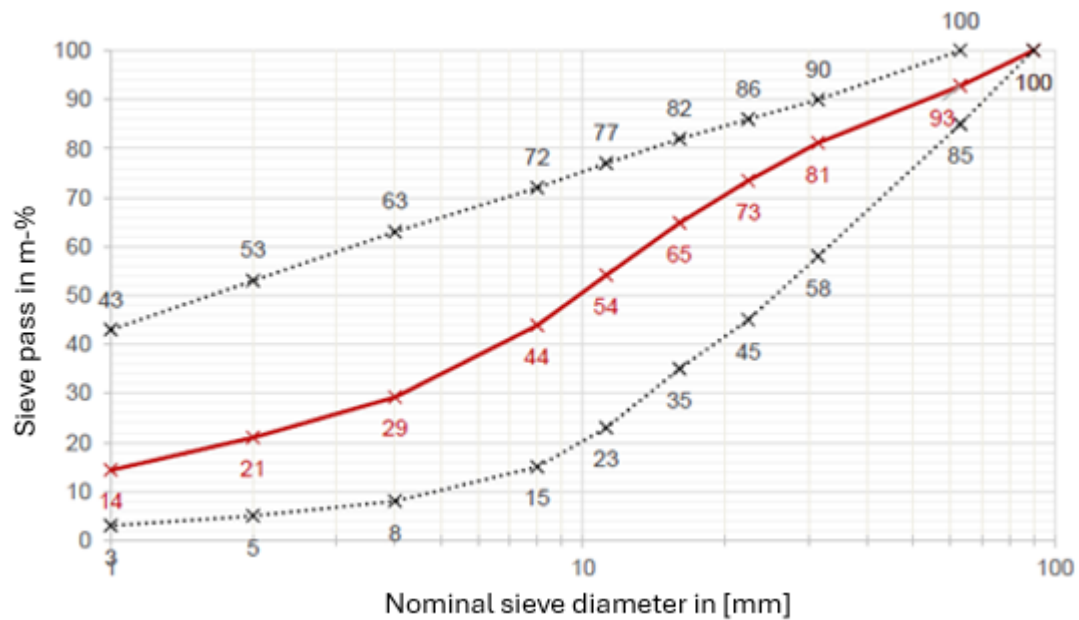


Figure 20: Test-determined grading curve of the recycle sample RB II, 0/63, U7, U-A

Table 10: Particle size distribution recycle sample RB II, 0/63, U7, U-A

Sieboffnung [mm]	Rückstand [g]	Rückstand [%]	Siebdurchgang [%]
63	767	7,21	100,00
31,5	1.235	11,62	92,79
22,4	823	7,74	81,17
16	913	8,59	73,43
11,2	1.139	10,71	64,84
8	1.086	10,22	54,13
4	1.559	14,66	43,91
2	880	8,28	29,25
1	706	6,64	20,97
Siebpfanne	1.526	14,33	14,33
Summe	10.631	100,00	100,00



Figure 21: Evaluation of the residue in the analysis sieves of concrete recylate RB II 0/63, U7, U-A

- Concrete recylate RB III, 0/32, U-A

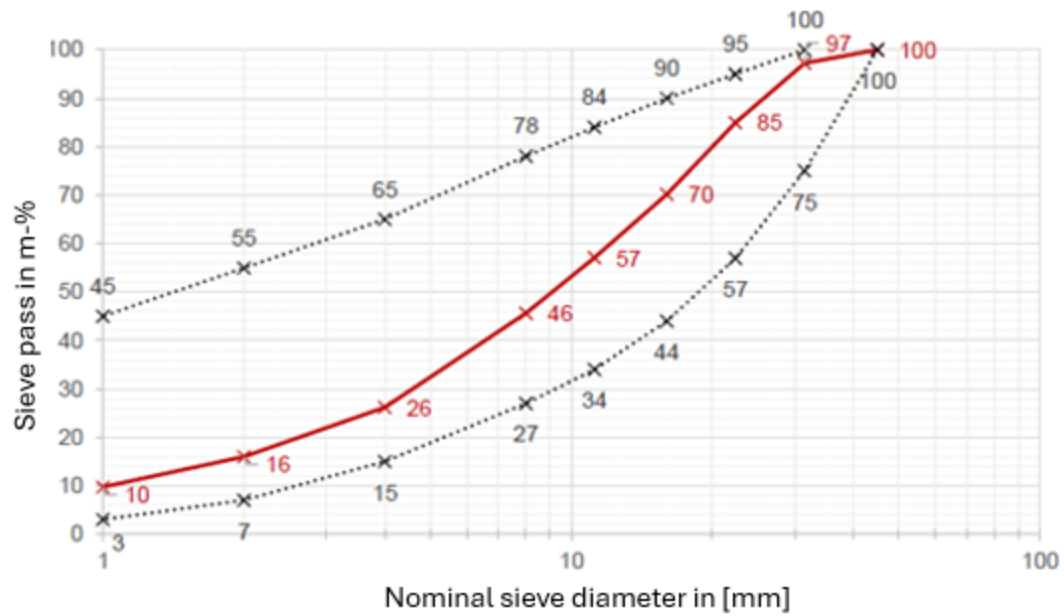


Figure 22: Test-determined grading curve of the recylate sample RB III, 0/32, U-A

Table 11: Particle size distribution recycle sample RB III, 0/32, U-A

Sieböffnung [mm]	Rückstand [g]	Rückstand [%]	Siebdurchgang [%]
31,5	272	2,76	100,00
22,4	1.212	12,29	97,25
16	1.462	14,82	84,96
11,2	1.286	13,04	70,14
8	1.133	11,49	57,10
4	1.916	19,42	45,61
2	1.004	10,18	26,19
1	619	6,28	16,01
Siebpfanne	960	9,73	9,73
Summe	9.864	100,00	100,00



Figure 23: Evaluation of the residue in the analysis sieves of concrete recycle RB III 0/32, U-A

- Building construction recycle RMH III 0/63, U-A

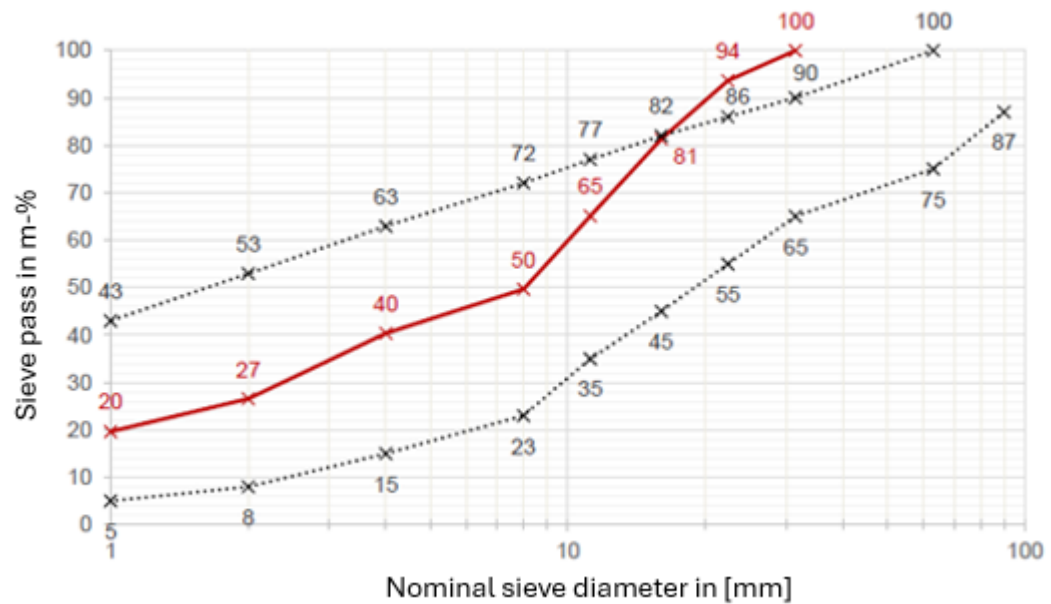


Figure 24: Test-determined grading curve of the recycle sample RMH III, 0/63, U-A

Table 12: Particle size distribution recycle sample RMH III, 0/63, U-A

Sieböffnung [mm]	Rückstand [g]	Rückstand [%]	Siebdurchgang [%]
31,5	623	6,35	100,00
22,4	1.212	12,34	93,65
16	1.594	16,24	81,31
11,2	1.514	15,42	65,07
8	913	9,30	49,65
4	1.354	13,79	40,35
2	679	6,92	26,56
1	756	7,70	19,95
Siebpfanne	1.173	11,95	11,95
Summe	9.818	100,00	100,00

The determination of the chloride content of the demolition material is particularly relevant due to the planned forced carbonation of the material samples. Chlorides bound in the material can be released in addition to the free chlorides due to the change in pH value and thus lead to increased corrosion potential of the recycled aggregate. The recycles listed were tested for their chloride content before and after forced carbonation in accordance with (OENORM EN 14629 2007) using acid digestion and potentiometric titration. In contrast to the sampling of the eluate, the acid digestion of ground recycle samples also shows chlorides bound in the cement paste.

A progressive carbonation process can lead to the dissolution of Friedel's salt and thus to an increase in the measurable chloride values.

Table 7: Overview of the chloride content of the recycled material samples in the non-carbonated state

Sample	Carbonation state	Chloride content
RB II, 0/63, U7, U-A	No carbonation	<0.05 M-%
RB III, 0/32, U-A	No carbonation	<0.05 M-%
RMH III, 0/63, U-A	No carbonation	<0.05 M-%

In addition to the classified recyclates from mixed, different sources described above, demolition material was specifically obtained from infrastructure structures in order to be able to use unmixed concrete demolition waste for sampling. This direct sampling makes it possible to assign used concrete directly to components and thus to the respective environmental exposures. For the further series of tests, it is important to identify recycled concrete containing chloride, particularly with regard to the material behavior of recycled concrete with chloride contamination in terms of its durability properties.

Direct sampling-Object 1: Demolition and new construction of a road bridge over a multi-track railroad line

- L13 Bockfließerstraße, Deutsch-Wagram, Austria
- 5-span, inclined reinforced concrete tee beam structure, L = 91 m, W = 9.40 m, h = 60 cm
- built: 1961, renovated: 1992
- Determination of compressive strength, carbonation depth, chloride content on 4 components with 2 test points each: supporting structure, edge beams, abutments and piers



Figure 25: Road bridge in Deutsch-Wagram, L13 Bockfließersstraße over the ÖBB before demolition

Samples were taken to determine the compressive strength (drill cores), the chloride content (drill dust samples) and the carbonation depth (phenolphthalein test on the newly produced fracture surface) before the structure was demolished. Figure 26 shows an overview of the sampling points for determining the described building material parameters and condition.

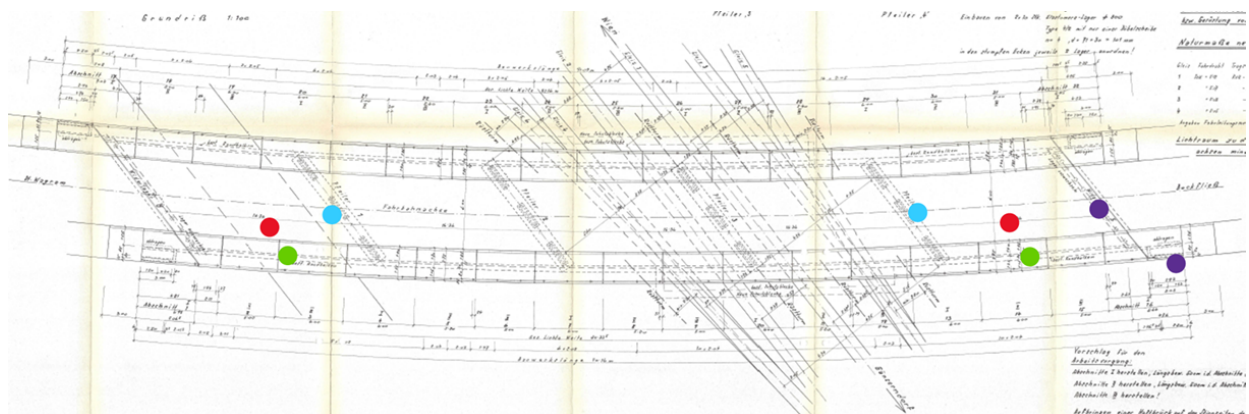


Figure 26: Ground plan of road bridge L13 with the marked sampling points. Supporting structure (red), edge beams (green), piers (blue), abutments (purple)

Material strengths

The present concrete compressive strengths of the structure were determined on 2 components before demolition (superstructure and edge beams). To test the strengths, cylinder samples were produced from drill cores (approx. $d=150\text{mm}$, $h=150\text{mm}$).



Figure 27: Compressive strength testing of cylindrical specimen

Table 7: Overview of the sample properties and test values of the compressive strength of superstructure and edge beam material

	Diameter [cm]	Height [cm]	Mass [g]	Force [kN]	Strength [N/mm ²]
Superstructure					
T1	14.30	14.63	5668.50	1334.60	83.10
T2	14.28	14.52	5721.00	1298.70	81.09
T3	14.30	14.69	5747.50	1194.50	74.37
T4	14.29	14.72	6056.50	1227.80	76.55
				MEAN	78.78
Edge beam					
R1	14.26	14.68	5240.50	596.40	37.34
R2	14.27	14.66	5194.50	486.10	30.39
R3	14.27	14.65	5259.00	590.10	36.90
				MEAN	34.88

The compressive strengths achieved in the tests are far above those of the concrete types used according to the planning documents, this applies above all to the load-bearing structure material (B 300, corresponds to C 25/30 according to the existing planning documents).

Carbonation state

The carbonation depth was determined on newly produced broken concrete surfaces using a phenolphthalein test.

Table 7: Results overview of the determined carbonation depths according to component and exposure

	Location	Carbonation depth 1 [mm]	Carbonation depth 2 [mm]	Carbonation depth 3 [mm]	Mean [mm]
Superstructure	-	11.87	12.04	12.29	12.07
Superstructure	-	8.54	5.18	7.97	8.22
Edge beam	-	10.47	11.68	11.99	11.38
Edge beam	-	12.31	14.87	14.21	13.80
Abutment	Protected	16.68	17.35	16.87	16.97
Abutment	Exposed	9.88	8.47	11.12	9.82
Pier	Exposed	10.31	9.78	9.44	9.84
Pier	protected	4.52	6.14	5.13	5.26

Chloride contents

The chloride content of the component material was tested on drill dust samples from a total of 3 depth levels and 3 drill holes per test site. Using acid digestion and potentiometric titration, none of the samples tested showed a value higher than 0.05 M-% in relation to the cement mass.

Direct sampling-Object 2: Production of concrete topping layers for cross-sectional reinforcement and edge beam renovation of several bridge structures

- Renovation of Vienna west exit, Nikolai bridges
- built in 1967
- Construction of new edge beams over a length of 2.1 km
- Cross-section additions amounting to approx. 850 m³
- Sampling of demolished concrete from the edge beam area and cross-sectional sample of edge beam incl. decking and section of cantilever slab (Figure 28)



Figure 28: Renovation of the bridge support structures of the Vienna West Exit with demolition of the edge beams and production of a concrete topping (left), edge beam cross-section for sampling in the FHCW laboratory (right)

3.7 Development and optimization of suitable concrete mixtures

3.7.1 Mix design of the mortars in Brazil

To evaluate the effects of recycled aggregates (RA) and carbonated recycled aggregates (CRA) in mortars, five different mixtures were considered. The mix proportions for the mortars are shown in the Table below. The reference mix is composed by 100 % natural aggregates. To address the effects of using carbonated and non-carbonated recycled aggregates, there were use replacement rates of 50 % and 100 % of the natural aggregates (NA) by the recycled aggregates in each condition in respect to carbonation. The w/c ratio was adjusted for the 100 % RA condition, as it contains no natural aggregates and carbonation tends to decrease the porosity of the recycled aggregates by forming CaCO_3 , thereby improving their workability. As a result, the w/c ratio was set at 0.65, based on a preliminary workability evaluation of 100 % RA mortars, which achieved a satisfactory slump value of 155 mm. No superplasticizer was added to the mixtures.

Table 13: Mortar mix design (values in kg/m^3 of mortar)

	Cement	NA	RA	Water
Ref	479.67	1439.02	-	311.79
50RA	479.67	719.51	674.84	311.79
100RA	479.67	-	1349.67	311.79
50CRA	479.67	719.51	684.46	311.79
100CRA	479.67	-	1368.92	311.79

The CRA was carbonated for 7 days in an environmental chamber under 5 % CO_2 , 30 °C and 65 % relative humidity (as detailed in task 3.3. of this report).

3.7.2 Concrete mix designs at FHCW

Recycled concrete mixtures were produced using the recycled samples described above. For this purpose, different proportions of the natural aggregate were replaced with recycled aggregates. A total of 3 different mixing ratios were sampled for each sample type (recyclate type). The individual mixing ratios of the natural and recycled aggregates were aimed at uniformly for all sample types. It should also be noted that a maximum grain size of 22mm has been specified for sample production. In the first step, the suitability of the mixtures is assessed on the basis of compressive strength tests on cubes with an edge length of 150 mm. In addition, mortar prisms (40x40x160 mm) are produced for bending tensile and compressive strength tests.

The materials listed in Table 14 and Table 14Table 15 were used to prepare the described concrete samples:

- Cube specimens, 150 x 150 x 150 mm

Table 14: Materials for the production of concrete samples for strength testing on 150 mm cubes

Aggregates	Concrete recycled aggregates, grain size 0 – 22 mm Natural aggregates, grain size 0 – 22 mm
Binder	CEM II / B-M (S-L) 32.5 R
Additives	none
	water

- Prism specimens, 40 x 40 x 160 mm

Table 15: Materials for the production of concrete samples for strength testing on prisms (40 x 40 160 mm)

Aggregates	Concrete recycled aggregates, grain size 0 – 4 mm Natural aggregates, grain size 0 – 4 mm
Binder	CEM II / B-M (S-L) 32.5 R
Additives	none
	water

The cube specimens were produced with different replacement rates of the natural aggregate, based on the Austrian standard ÖNORM B 4710 (*Beton - Festlegung, Eigenschaften, Herstellung, Verwendung und Konformität - Teil 1: Regeln zur Umsetzung der ÖNORM EN 206 für Normal- und Schwerbeton*), whereby the sand fraction was separated at 0 – 4 mm. This grain fraction was not used for the cube samples, to avoid the influence of the fine fraction on the material properties. Their influence is investigated in a separate sample series with small scale prisms containing only fine aggregates.

The ÖNORM B 4710 defines permissible maximum replacement rates concerning the use of recycled aggregates replacing natural grains in concrete mixes for structural purposes. These depend on the grain fractions and particle sizes. Lower replacement rates are allowed with fine grain fractions, compared to coarse grains. Based on the available recycled aggregates, taking the limitations according to ÖNORM B 4710 into account, the following replacement rates were defined for the experimental testing at FHCW:

- Cube testing:
 - o Specimens were produced with recycling aggregate rates according to Table 16 and 3 pieces per mix design.

Table 16: Defined aggregate replacement rates for cube specimens production using RC-mixes

Replacement of natural aggregates by recycling material from 4 to 22 mm particle size	replacement rate
Type C-0	0 %
Type C-1	19 %
Type C-2 (replacement rate according to Austrian standard)	38 %
Type C-3	57 %

Table 15: Test values of the reference concrete mix, indicating the average compressive strength of 3 samples

	Specimen 1 [N/mm ²]	Specimen 2 [N/mm ²]	Specimen 3 [N/mm ²]	Mean [N/mm ²]
Type C-0	35.8	31.6	34.4	33.9

Table 16: Test values of the recycled concrete formulations, indicating the average compressive strength of 3 samples

	Specimen 1 [N/mm ²]	Specimen 2 [N/mm ²]	Specimen 3 [N/mm ²]	Mean [N/mm ²]
Type C-1	30.1	35.9	36.9	34.3
Type C-2	34.2	33.8	29.9	32,6
Type C-3	28.6	31.0	24.7	28,1

The strength tests carried out show a possible correlation between the compressive strength and the recycled content in the concrete composition. The deviation from the reference recipe with purely natural aggregates is weak from a structural point of view. The failure mechanism and the fracture pattern of the RC concrete specimens can be described as equally insignificant.



Figure 29: Cube specimen type C-3 after compressive strength testing

- Prism testing:
 - o Specimens were produced with recycling aggregate rates according to Table 16 and Table 17 and 3 pieces per mix design.

Table 17: Defined aggregate replacement rates for prism specimen production using RC-mixes

Replacement of natural aggregates by recycling material from 0 to 4 mm particle size	replacement rate
Type P-0	0 %
Type P-1	50 %
Type P-2	100 %



Figure 30: Fracture surface of prism specimen type P-1 after bending tensile testing

3.8 Replacement of Portland cement by new binders

One of the main sustainability focus of this project was the determination of the potential of recycled aggregates to absorb and fix CO₂, then a carbon capture strategy to be considered by the building sector. Also, it was investigated how the use of carbonated recycled concrete aggregates (with and without chloride content) affects the behavior of mortar and concretes in terms of short term and durability related properties. Therefore, up to now, the studies of mortar and concretes only considered conventional concretes, without supplementary cement replacing materials. Only after the results of this report, it is possible to consider how a partial replacement of cement by mineral admixtures can bring further contributions to performance of mortars and concretes containing carbonated recycled aggregates.

3.9 Determination of the time depending behavior

Behavior of mortars produced with fine recycled (RA) and carbonated recycled aggregates (CRA) in Brazil.

The results for workability, water absorption, and bulk density of the mortars produced with NA, RA, and CRA are presented in the Figure 31. The reference samples exhibited the lower water absorption (11.26 %) and higher density (1.90 g/cm³) compared to the mortars with RA and CRA. The slump value for the Ref mortars was 223.33 mm, considered the baseline (100 %) for this property. Regarding RA and CRA mortars, those made with CRA exhibited lower water absorption values, both for 50 % (11.90 %) and 100 % (13.31 %) replacement rate, compared to the non-carbonated recycled aggregates (12.58 % for 50 % and 13.40 % for 100 % replacement rates). This indicates that accelerated carbonation had a positive impact on the properties of the recycled aggregates. Furthermore, RA mortars showed a decreased slump compared to the reference mortars (99.25 % and 98.51 % for partial and total replacement rates). In contrast, CRA mortars demonstrated an increased slump for both replacement rates (102.24 % for 50 % and 103.73 % for 100 % replacement rates), suggesting that CO₂ curing was beneficial for enhancing the properties of recycled aggregates.

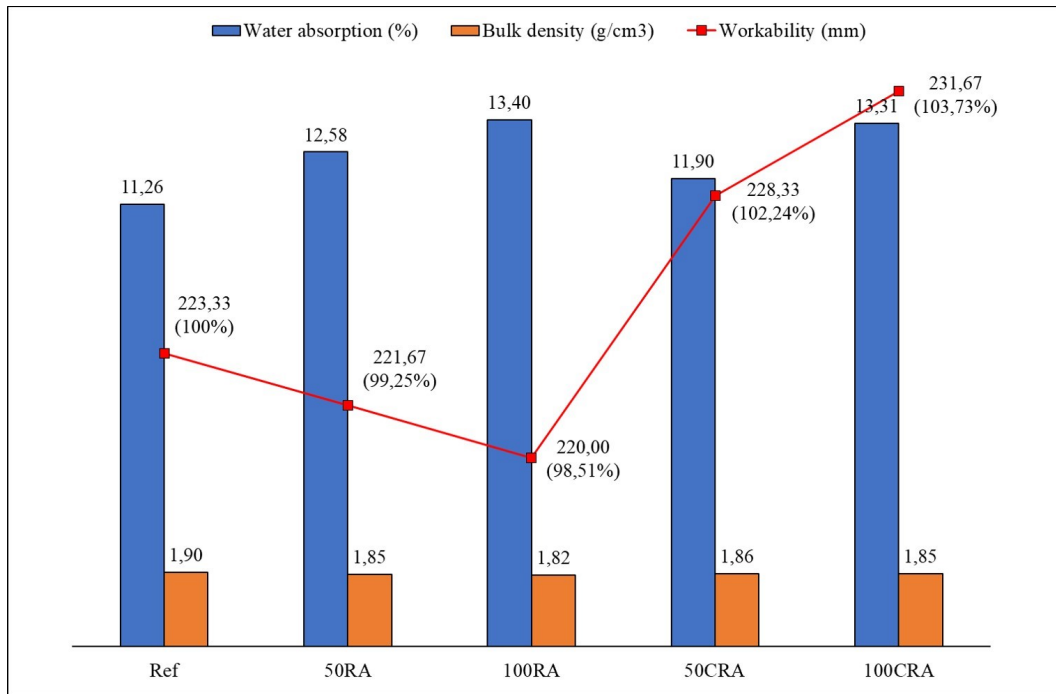


Figure 31: Bulk density, water absorption and slump of mortars

The compressive strength results at 7 and 28 days for the mortars are presented in Figure 32. At both ages, the results indicate that mortars made with recycled aggregates achieved higher compressive strength values compared to the reference mortars.

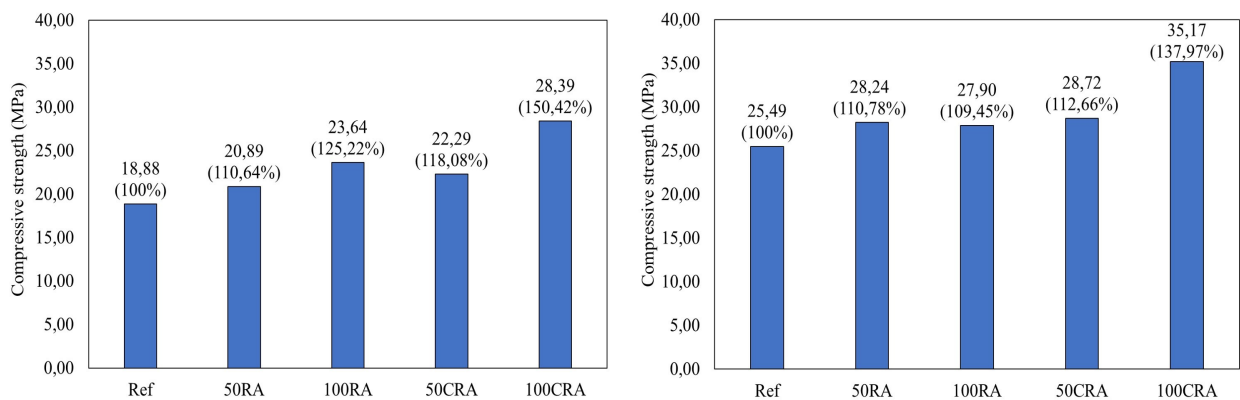


Figure 32: Compression strength of mortar with RA and CRA at 7 (left) and 28 (right) days

4 Determination of the durability parameters of concretes with chloride-containing aggregates

4.1 Corrosion of steel in chloride-containing RC eluate

4.1.1 General

To assess the worst-case scenario, corrosion tests were conducted on concrete steel in the eluates of recycled aggregates. The steel used was B500A type, as defined in DIN EN 488-1, with a diameter of 10 mm. The samples were cut to a length of 15 cm and their surfaces degreased with isopropanol. The eluates were obtained from recycled aggregates (see Sections 3.2 and 3.3).

4.1.2 Setup and procedure

The experimental setup used a three-electrode configuration to measure the current flow between the working electrode (WE) and the counter electrode (CE), as well as the WE's electrode potential, using a reference electrode (RE) and a potentiostat for control.

In the corrosion tests, the reinforcing steel acted as the WE and was immersed 60 mm deep in the electrolyte solution (RC eluate). The CE was a titanium strip coated with mixed metal oxide (Ti-MMO) and the RE was a saturated calomel electrode (SCE). The test setup is shown in Figure 33.

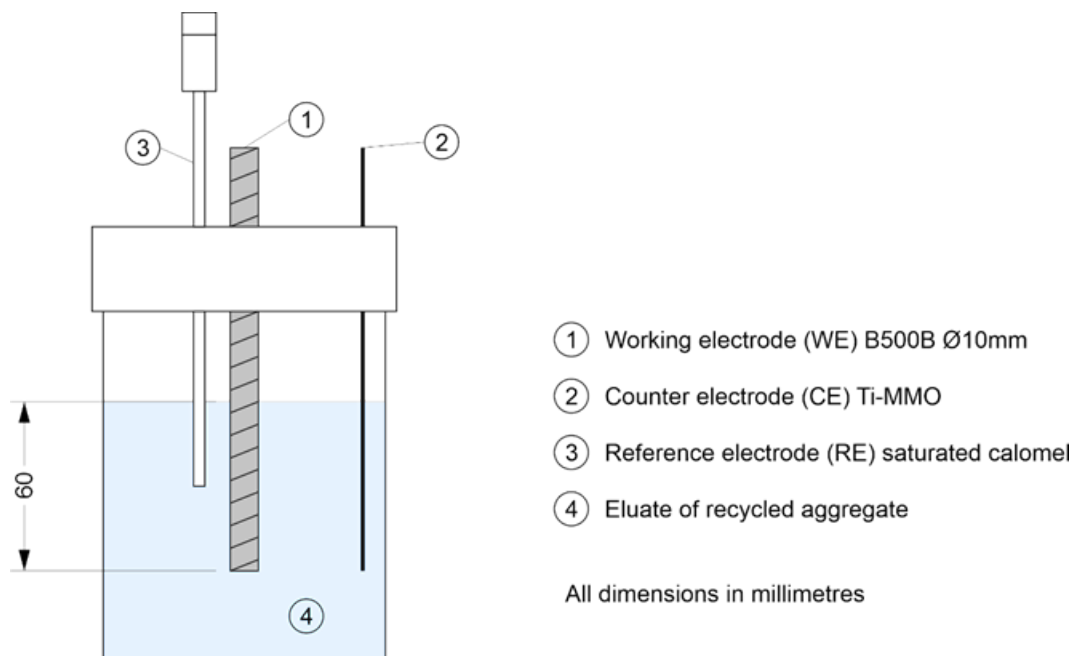


Figure 33: Setup for corrosion tests in eluates.

The free corrosion potential (E_{cor}) of the steel in the electrolyte was determined prior to the corrosion tests. Afterwards, the samples were polarised by 100 mV in the anodic direction for 24 hours, starting from the free corrosion potential. The results were evaluated by integrating

the current density (i) over the test period (t). The integral $\int i(t)dt$ provided the specific current flow of the steel samples, thereby offering insights into the corrosion intensity during the corrosion test.

4.1.3 Results

Figure 34 shows E_{cor} of the steel samples after three hours in various electrolytes. The lowest potentials are found in the LS and CI eluates, followed by TK and TS. Conversely, OC and CC exhibit relatively high potentials. Overall, the potentials correlate well with the chloride content (see Chapter 3.5), with higher chloride content leading to a more negative potential. An exception is the LS eluate, which has a very low chloride content yet a very negative potential.

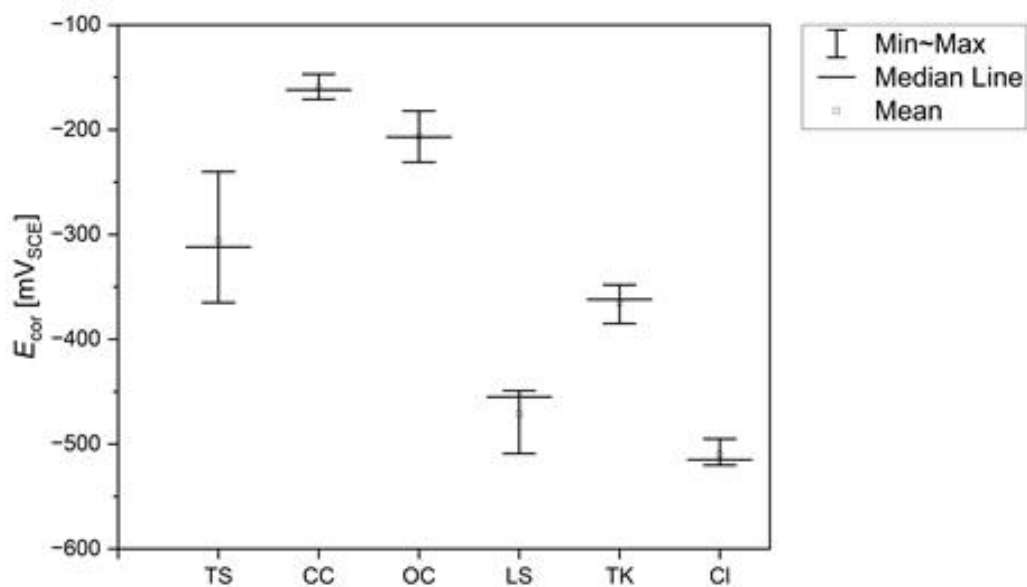


Figure 34: Free corrosion potential E_{cor} of steel samples in various electrolytes.

Figure 35 shows the specific current flow obtained from the i - t curves for the various eluates. TS exhibits the highest specific current flow, followed by TK, CI and LS. Severe corrosion was observed on the specimens in all four cases. By contrast, the two eluates CC and OC exhibit only a very low specific current flow.

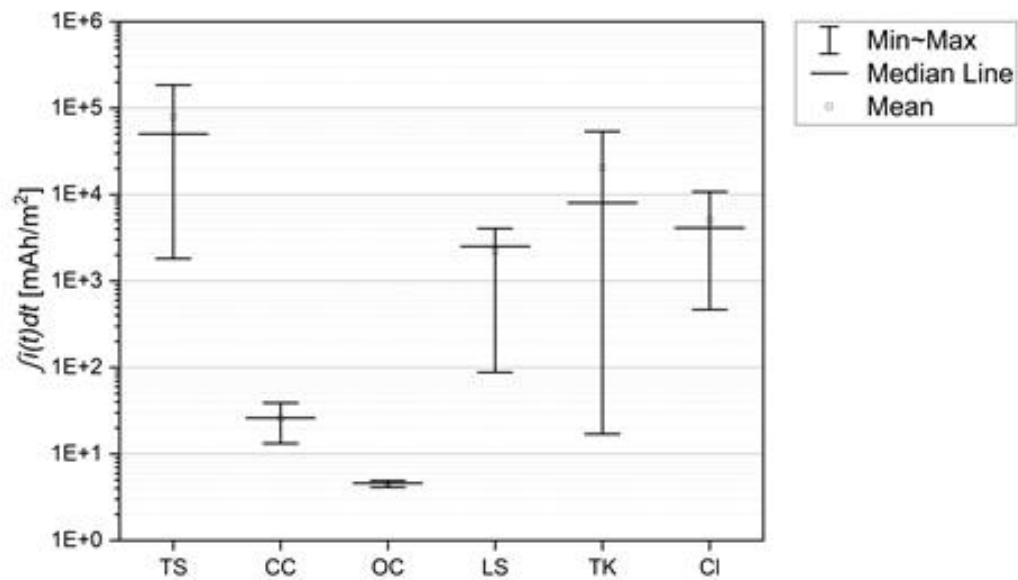


Figure 35: Specific current flow $\int i(t) dt$ of the current density-time curves for steel in eluate.

Despite the high pH values between 12 and 13 of the eluates, measured after the experiment, hardly any stable passive layers were formed, as no chlorides are chemically bound in comparison to RC.

4.2 RC-specimen corrosion tests

4.2.1 General

As corrosion behaviour in concrete generally differs significantly from that in a solution, tests are also carried out on concrete cylinders containing 100 % RCA and axially arranged reinforcing steel.

To this end, composite specimens comprising reinforcing steel (see Section 4.1) and concrete containing chloride-bearing aggregate (see Section 3.3) were produced. These specimens were then placed in a saturated calcium hydroxide solution and polarised using a potentiostat.

4.2.2 Setup and procedure

The test setup (see Figure 36) and the procedure are similar to that used for the corrosion tests in the eluate (see Chapter 3.1). A three-electrode setup was also used, with the procedure being analogous to that used in the eluate tests. First, E_{cor} was determined, and then the anodic polarisation was applied at 100 mV.

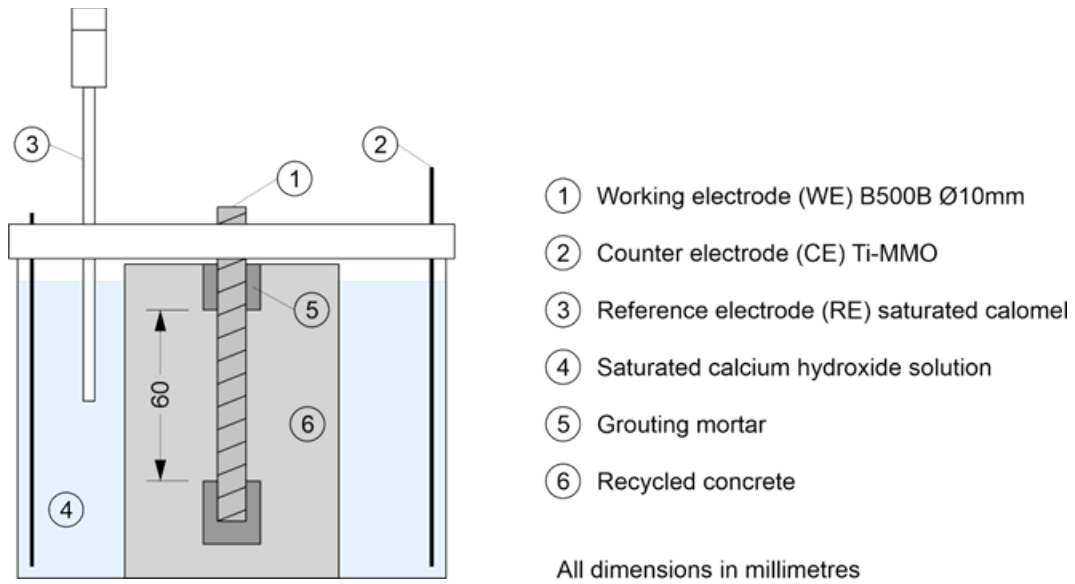


Figure 36: Setup for corrosion tests with RC specimens.

4.2.3 Results

Figure 37 shows E_{cor} of the RC samples after three hours in various electrolytes. The mean values of the potentials of the RC specimens TS, CC, OC and LS are almost identical at -300 to -350 mV. Only the TK and CI specimens show significantly more negative potentials.

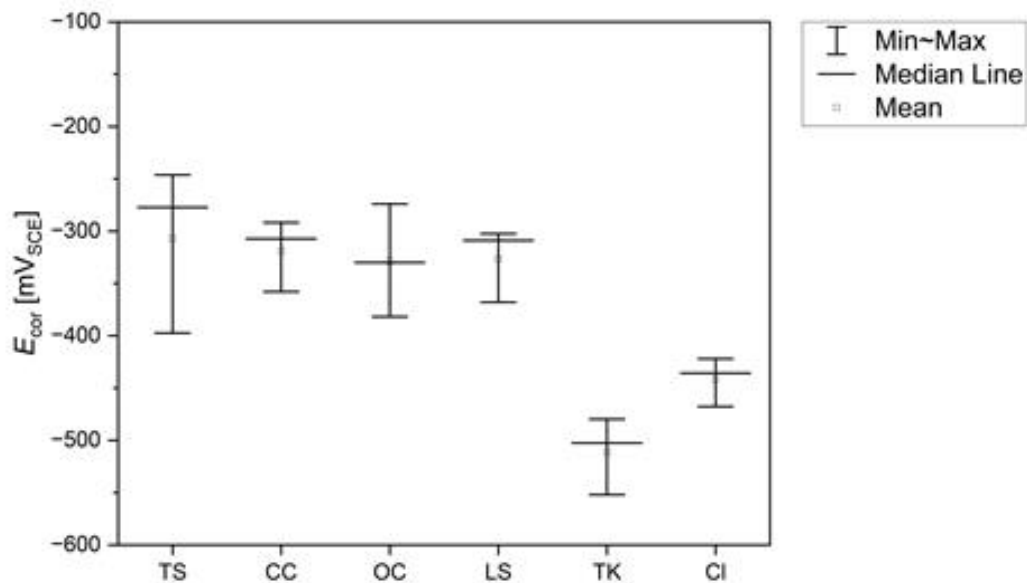


Figure 37: Free corrosion potential E_{cor} of RC samples in saturated calcium hydroxide solution.

Figure 38 shows the specific current flow obtained from the $i-t$ curves for the various RC samples. The mean values of specific charge flow range from 30 to 143 mAh/m². The lowest mean values, at 30 to 38 mAh/m², are found in TK, LS, CC and OC, while the highest are found in TS and CI. Therefore, the corrosion activity of the steel in the RC specimens remains

at a very low level throughout. According to (Bertolini 2013), a corrosion current in concrete of up to 1 mA/m² (or 0.1 µA/cm²) is considered negligible for the durability of a structure, which corresponds to a limit value of 24 mAh/m² in a 24-hour test.

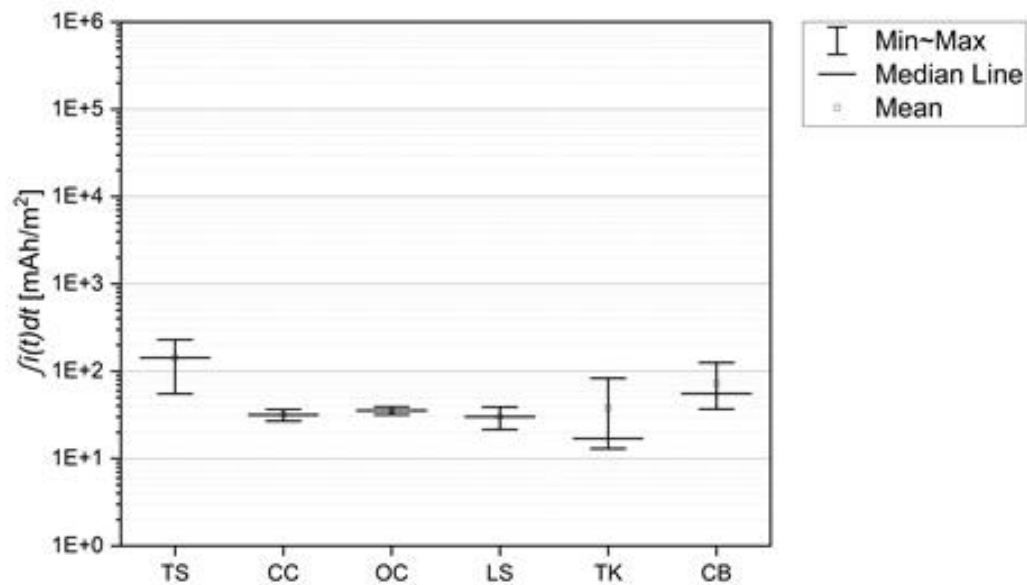


Figure 38: Specific current flow $\int i(t)dt$ of the current density-time curves for steel in RC.

Taking into account the maximum current density, corrosion rates of up to 81 µm/a can be derived from the corrosion tests on the RC samples. However, this approach tends to significantly overestimate the corrosion rate due to individual current peaks, and therefore represents an unlikely extreme case for the RC samples analysed here. In addition, the RC samples have a high water content and were aerated from above. This increases the conductivity of the concrete, improves the oxygen supply and increases the corrosion rate. Under dry conditions, however, the corrosion rate is reduced further.

When the specific current flow for steel in concrete is compared with the chloride content of the RCA (see Chapter 3.2), it can be seen that low chloride content generally leads to lower specific current flow. The RCA TS is an exception, however, as it results in an extremely high charge flow in both the eluates and the concrete, making evaluation more difficult.

4.3 RC-specimen corrosion tests with chloride containing and carbonated aggregates

4.3.1 Material for Corrosion Tests

Recycled Concrete Aggregates (RCA)

To produce the recycled concrete aggregates (RCA) used in the corrosion tests, two parent concrete mixtures were prepared, each incorporating three levels of chloride content (1.5 wt.-%/c, 2.0 wt.-%/c, and 3.0 wt.-%/c), with chloride added via the mixing water. The concretes were composed of Portland cement (CEM I 42.5 N) and blast furnace cement (CEM III/A 42.5 N), with a water-to-cement ratio of 0.55 and a grading curve A/B16. The material volumetric calculation is shown in Table 18.

Table 18: Composition of the parent concretes (CEM I 42.5 N and CEM III/A 42.5 N) for the production of RCAs

Material	Mass	Density	Volume
[-]	[kg/m ³]	[kg/dm ³]	[dm ³]
CEM	350	3.1	113
Water	193	1	193
0/4	843	2.7	312
4/8	403	2.7	150
8/16	587	2.7	217
Air voids	-	-	15
NaCl	1.5/ 2.0/ 3.0 wt.-%/c Cl		
Total	2376	2.376	1000

Following the casting process, the samples were stored in a climate chamber (20°C, 98 % RH) for 28 days, followed by approximately one year of storage under laboratory conditions (20°C, 55 % RH). Subsequently, the concretes were crushed using a jaw crusher and sieved to a final grain size of A/B8. The density, water absorption, and chloride content of the RCA produced from chloride-containing mixtures were determined. The results of the chloride analysis are presented in Figure 21.

Table 19: Results of the chloride analysis of the RCAs

Material	Chloride Concentration (photometric)	Chloride Concentration (eluate 1:10 (IC))
[-]	[wt.-%/c.]	[wt.-%/c.]
CEM I 1.5 wt.-%/c	1.43	0.85
CEM I 2.0 wt.-%/c	2.11	0.75
CEM I 3.0 wt.-%/c	2.23	1.83
CEM III 1.5 wt.-%/c	1.54	0.96
CEM III 2.0 wt.-%/c	1.58	1.28
CEM III 3.0 wt.-%/c	2.14	1.91

Recycled Concrete (RC)

The RCs used for the corrosion test specimens were produced using chloride-contaminated RCAs with a maximum particle size of 8 mm (0.31 in, A/B8), combined with the same types of cement as in the original parent mixes (CEM I 42.5 N and CEM III/A 42.5 N). A constant water-to-cement ratio of 0.55 was maintained. The composition of the concretes are presented in Table 20.

Table 20: Composition of the RCs (CEM I 42,5 N und CEM III/A 42,5 N) for the production of the corrosion specimens

Material	Mass	Density	Volume
[-]	[kg/m ³]	[kg/dm ³]	[dm ³]
CEM	360	3.1	116
Water	198	1	198
0/4	1011	2.2	456
4/8	545	2.5	215
Air voids	-	-	15
Total	2114	2.114	1000

4.3.2 Testing Methods for Corrosion Tests

For the corrosion tests, prismatic specimens with dimensions of 200 mm × 150 mm × 100 mm (7.87 in × 5.91 in × 3.94 in, L × W × H) were prepared. One out of three specimens per series was equipped with a silver/silver chloride (Ag/AgCl) reference electrode comprising two anodes. The anodes consisted of two B500 steel rebar sections welded to V4A stainless steel wires to prevent crevice corrosion, with concrete covers (dc) of 10 mm and 20 mm, respectively. The cathodes were made of titanium mixed metal oxide (TMMO) strips. The surface area ratio of anodes to cathode was 1:15. The silver/silver chloride reference electrodes were embedded at a depth of 15 mm and at a distance of approximately 30 mm from the steel anodes. Additionally, an electrolyte conductor made of V4A stainless steel wires was installed at depths ranging from 5 mm to 45 mm in 10 mm intervals. A schematic illustration of the corrosion specimen setup is provided in Figure 2.

The specimen designation is based on the binder type (CEM I / CEM III) and the chloride addition amount used during the production of the parent concretes for the RCAs (0.0 wt.-%/c., 1.5 wt.-%/c, 2.0 wt.-%/c, 3.0 wt.-%/c). The actual chloride contents (photometric and eluate 1:10) of the RCAs are presented in Table 20.

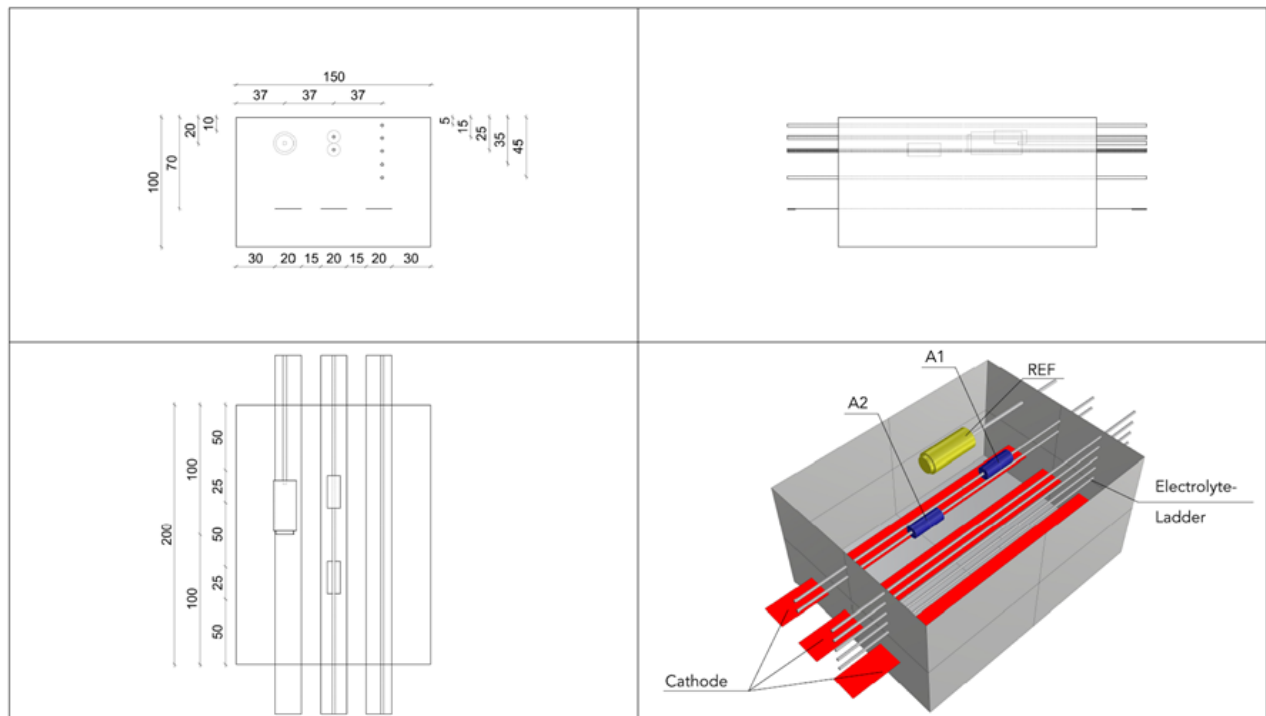


Figure 39: Specimen with three-electrode setup for the corrosion investigations, bottom right: Anodes in blue; cathodes in red and silver/ silver chloride electrodes in yellow

Corrosion Potential

The free corrosion potential was measured between the anodes A1 and A2 and the silver-silver chloride reference electrodes with a Potentiostat (Gamry Reference 600+).

Corrosion Current Density

Based on the specimen configuration, element currents between the anodes A1 ($d_c = 10 \text{ mm}$) / A2 ($d_c = 20 \text{ mm}$) and the cathodes can be recorded non-stationarily using ZRA measurements (Zero Resistance Ammeter), conducted with a Gamry Reference 600+ potentiostat, in a two-electrode setup as described in (Becker und Dauberschmidt 2020). To ensure that the cathodes could be shared by both anodes, the anodes and cathodes were short-circuited only during the measurement intervals. To avoid polarisation effects within the system, a minimum interval of 24 hours was maintained between measurements involving the two anodes.

4.3.3 Corrosion Tests

The measurements presented in the following results were conducted on specimens that had been conditioned in a climate chamber up to an age of 108 days (20°C , 98 % RH).

Corrosion potential

The corrosion potentials of both anodes (A1 and A2) were measured using a silver/silver chloride reference electrode. The decrease of the negative corrosion potentials, using an example of anode A1, illustrated in Figure 40 suggests, a decreasing corrosion activity in the concretes containing chloride-containing RCAs versus time.

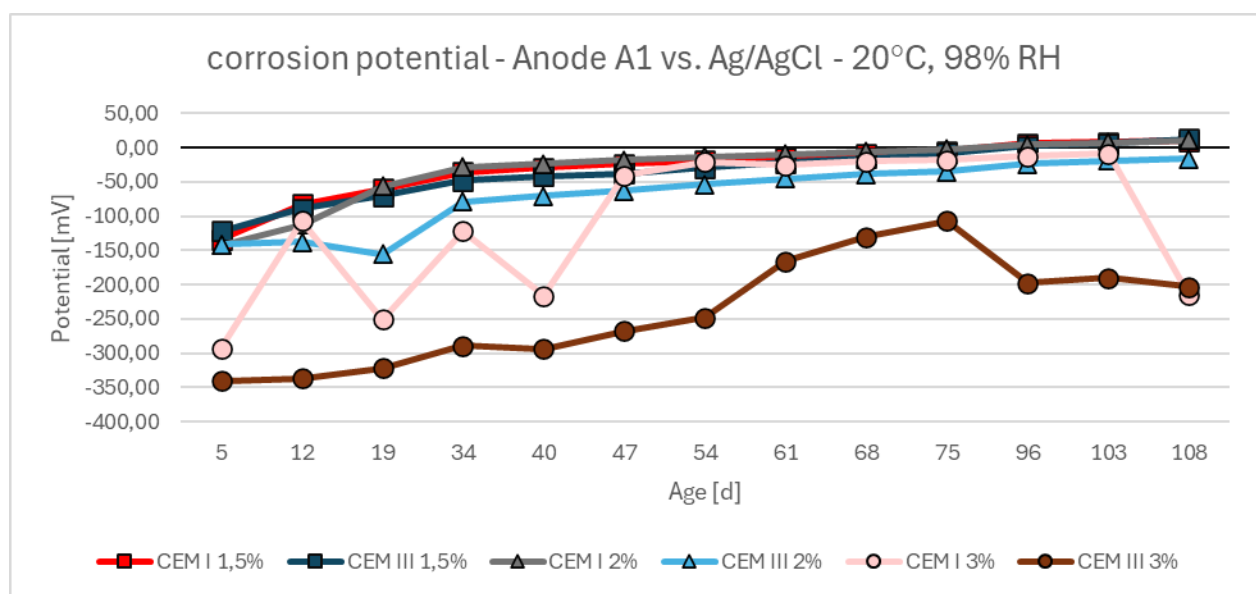


Figure 40: Corrosion Potential of Anode A1 ($d_c = 10 \text{ mm}$) vs. Silver/Silver Chloride reference electrode versus time (exposure conditions: 20°C , 98 % RH)

Despite storing the samples under corrosion-promoting environmental conditions to an age of 108 days (20°C, 98 % RH), the free corrosion potentials were not more negative than -250 mV at a specimen age of 54 days (Anode A1, $d_c = 10$ mm). Following the measurements until an age of 108 days of the specimen the corrosion potentials are still decreasing over time. The only specimen with a more negative potential is the CEM III specimen with 3 wt.-%/c. RCA (-203.6 mV_{Ag/AgCl}). The specimens of CEM I with 3 wt.-%/c. chlorides in the RCA parent-concretes also showed a more negative potential with -215,1 mV. Further research showed, that this datapoint eventually was corrupted due to an error in the measurement. Measurements before and after 108 days indicate corrosion potentials higher than -10 mV.

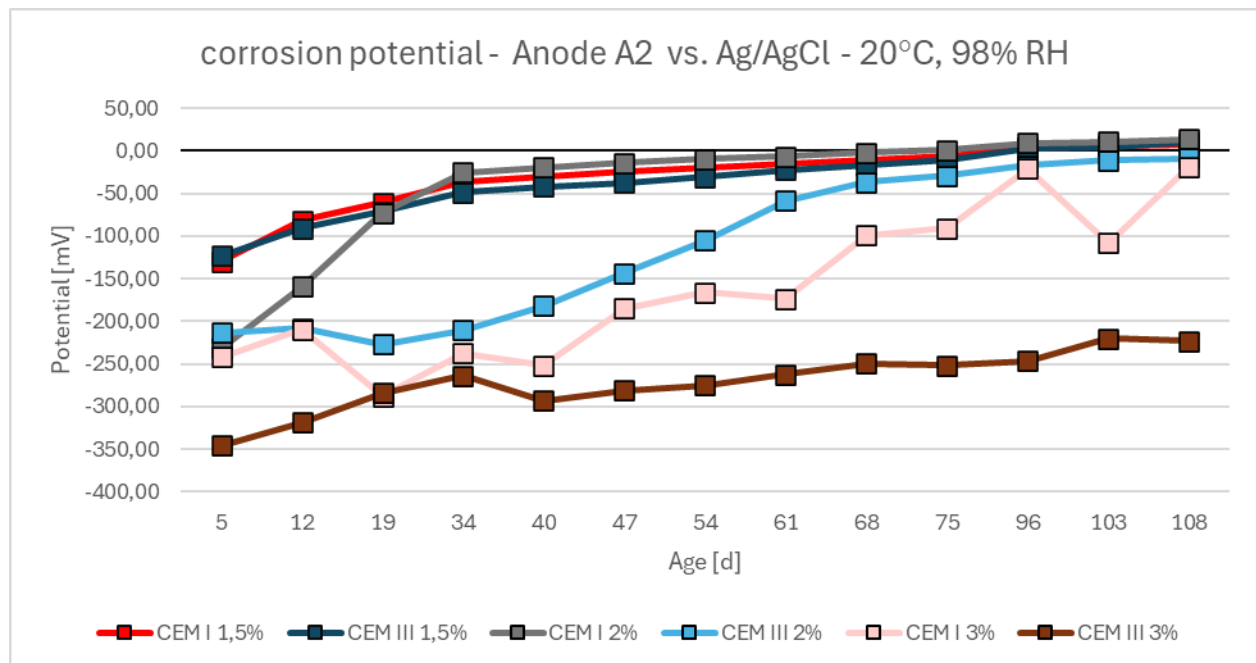


Figure 41: Corrosion Potential of Anode A2 ($d_c = 20$ mm) vs. Silver/Silver Chloride reference electrode versus time (exposure conditions: 20°C, 98 % RH)

Similar to the measurements on Anode 1, measurements were also conducted on the second anode A2 ($d_c = 20$ mm) versus the silver/silver chloride reference electrode. The results (see Figure 41) show a comparable trend to those of Anode A1. Due to the 10 mm higher concrete cover of Anode A2 compared to Anode A1, a slight delay is observed, resulting in more negative corrosion potentials over time. However, as with Anode A1, the corrosion potentials for A2 also stabilize - with the exception of the RC with RCA parent concretes with 3 wt.-%/c. chlorides and a corrosion potential of -223.5 mV - at values greater than -18.36 mV_{Ag/AgCl} after a specimen age of 108 days.

Corrosion current density

The measurement of the element currents was carried out directly after the short-circuit between anode and cathode in the ZRA setup. It was observed that after 300 seconds the element currents approached a constant value. The classification of the corrosion current densities in terms of steel cross-sectional loss, service life, and corrosion activity is based on (Arrhenius) (Cox) (Andrade & Alonso, 2001) (see Table 21).

Table 21: Ranges of Corrosion Current Density values related to the Annual Rebar Section Loss, the Lifespan of the Rebar and the Effect of Corrosion

Current Density (Arrhenius)	Annual Rebar Section Loss ¹⁾	Lifespan ²⁾ Rebar $\varnothing = 10 \text{ mm}$	Effect of Corrosion (Arrhenius 1989) (Cox 1997) (Andrade und Alonso 2001)
$[\mu\text{A}/\text{cm}^2]$	$[\text{mm}/\text{a}]$	$[\text{a}]$	$[-]$
≤ 0.1	≤ 0.001	> 100	negligible
$0.1 - 0.5$	$0.001 - 0.005$	$25 - 100$	low
$0.5 - 1.0$	$0.005 - 0.010$	$10 - 25$	moderate
> 1.0	> 0.010	≤ 10	high

¹⁾ Calculated according to Faraday for uniform corrosion.
²⁾ Cross-sectional loss $\leq 1 \%$ as the limit for the service life of a reinforcing steel with a diameter of 10 mm.

The element corrosion currents decrease versus time, similar to the corrosion potentials. The current densities (i_{corr}) of samples with chloride contents of the RCAs up to 2 wt.-%/c. and partly 3 wt.-%/c (CEM I) are within non-critical ranges ($< 0.1 \mu\text{A}/\text{cm}^2$) after 68 days. The results of the CEM III samples with 3 wt.-%/c chloride-containing RCAs indicate higher corrosion activity, decreasing versus time (see Figure 42 and Figure 43).

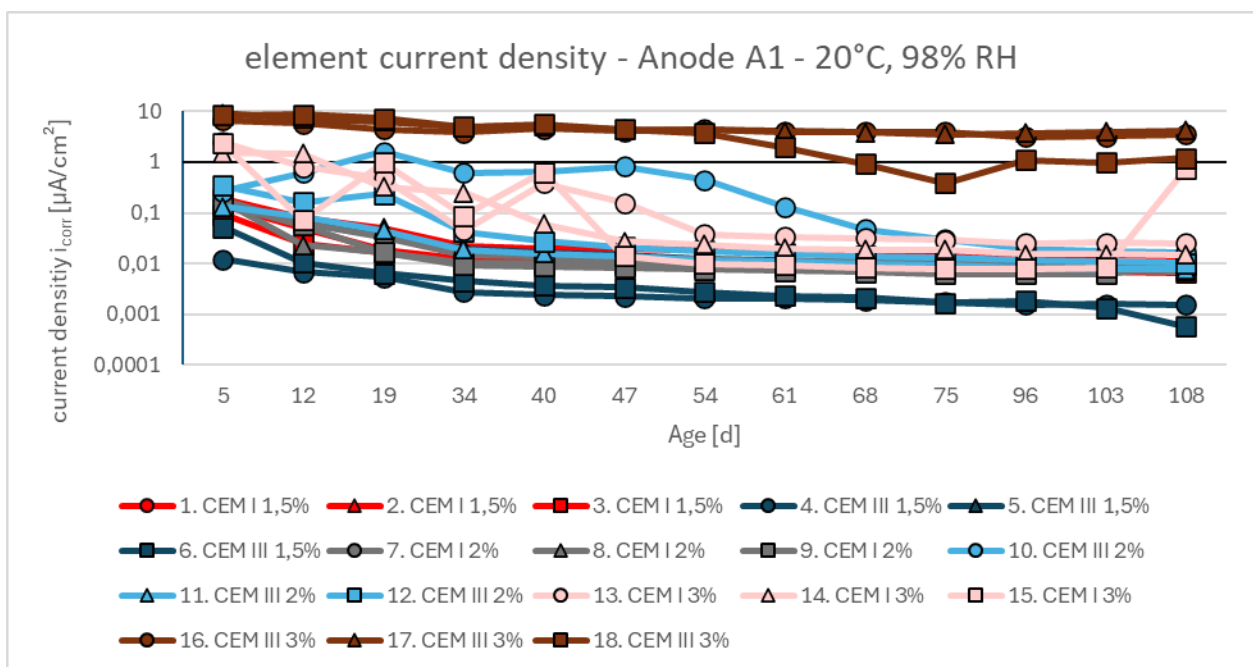


Figure 42: Element Corrosion Current Density of Anode A1 ($d_c = 10 \text{ mm}$) versus time (exposure conditions: 20°C/98 % RH)

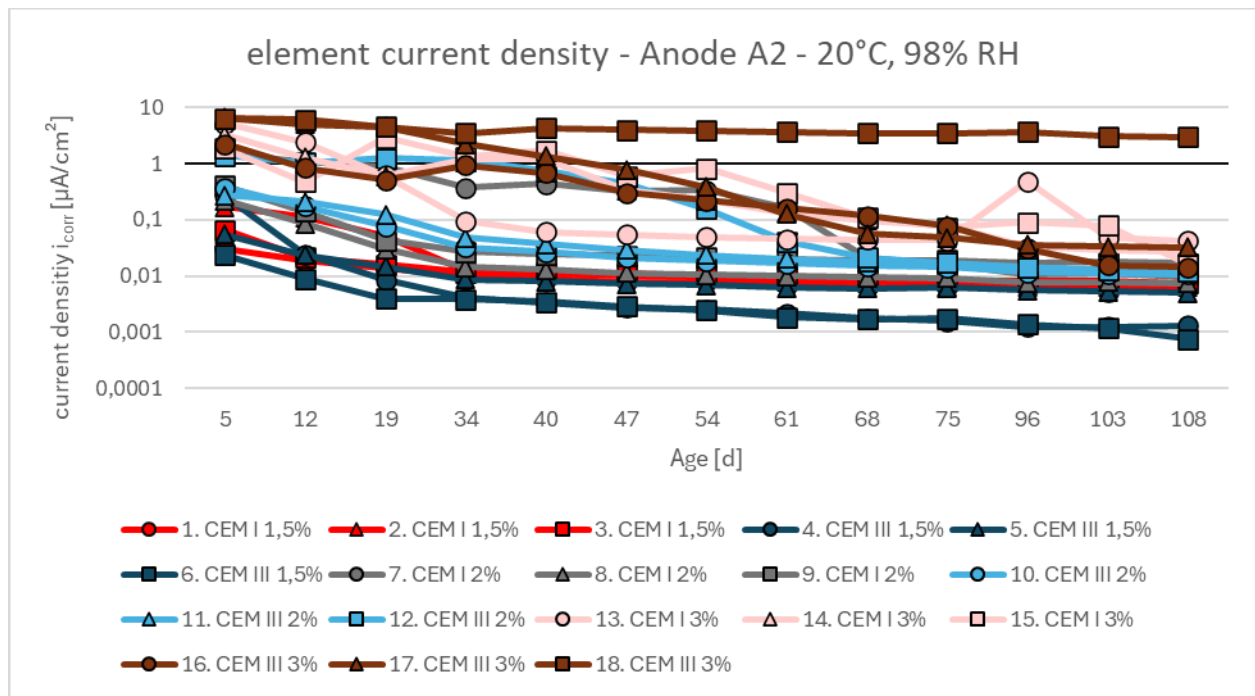


Figure 43: Element Corrosion Current Density of Anode A2 (dc = 20 mm) versus time (exposure conditions: 20 °C/ 98 % RH)

The investigations regarding the corrosion parameters show that, with the exception of specimens made from CEM III and those with an addition of 3 wt.-%/c. for the production of the RCAs, and a resulting chloride content of the milled RCAs of 2.14 wt.-%/c. (1,91 wt.-%/c. in the eluate), no detrimental reinforcement corrosion was observed on the anodes due to the recorded corrosion potentials and low corrosion current densities over time.

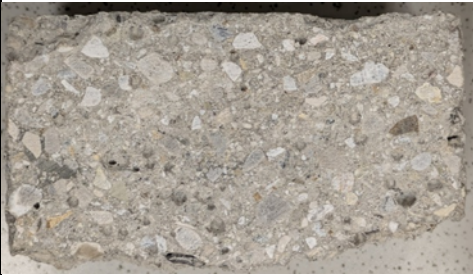
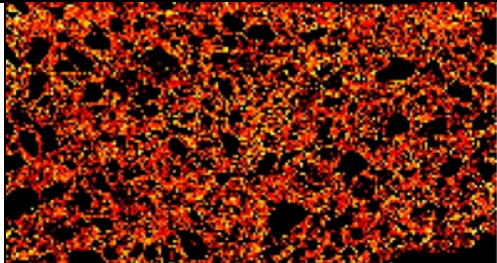
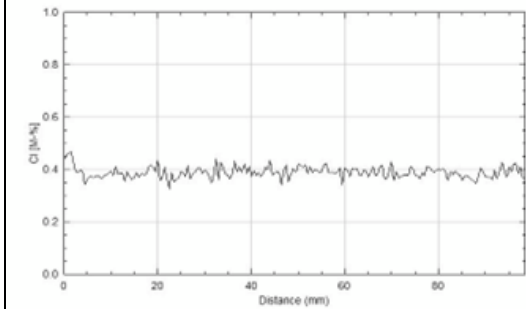
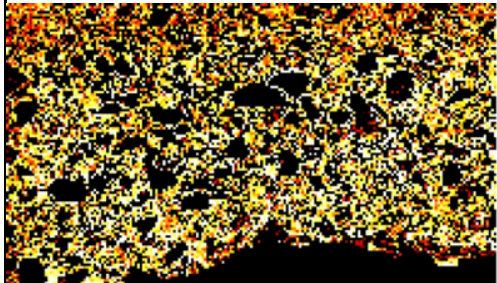
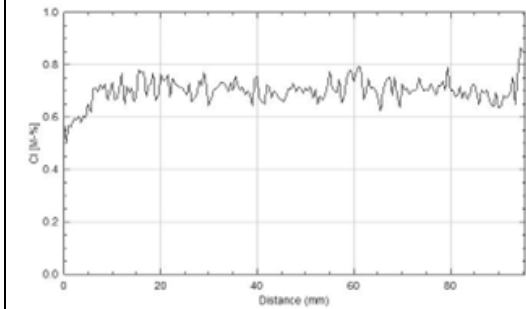
4.4 Determination of the time depending chloride redistribution

To investigate the time depending redistribution in RC of chlorides originating from RCAs, specimens were prepared with initial chloride contents of 1.5 wt.-%/c and 3.0 wt.-%/c. The volumetric composition was identical to that of the concretes presented in Table 20. The same RCAs and cements were employed in the mix design

Laser-induced breakdown spectroscopy (LIBS) was used as the method of analysis. LIBS enables the spatially resolved detection of element concentrations by focusing a high-energy laser pulse onto the sample surface, generating a plasma whose emitted light is analysed spectroscopically.

The measurements were performed on fractured prism specimen surfaces at an age of 7 days. A photographic documentation and the measurements of the specimen produced with CEM I are shown in Table 22, with CEM III in Table 23.

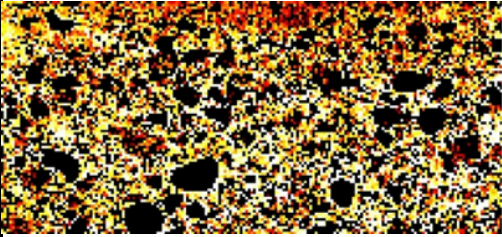
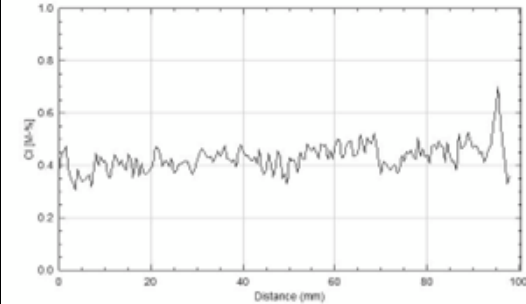
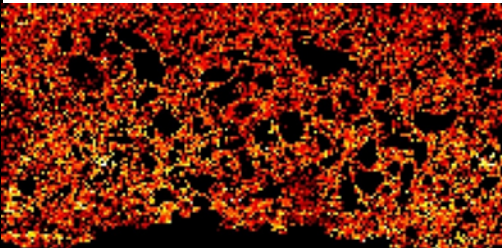
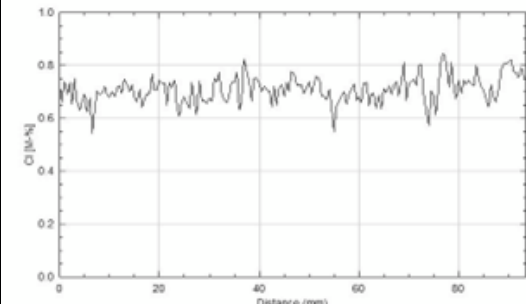
Table 22: LIBS Measurements (Chlorides) of the specimens produced with Ordinary Portland Cement

Specimen	LIBS Cl Measurements	
-	Photo	Chlorides
CEM I 1.5 %		 
CEM I 3.0 %		 

A comparison of the results obtained from specimens produced with CEM I and varying chloride contents reveals that an increased initial chloride level leads to higher chloride content within the RC-concrete. Furthermore higher scattering of the chloride content can be observed. So a more pronounced redistribution of chlorides from the RCAs.

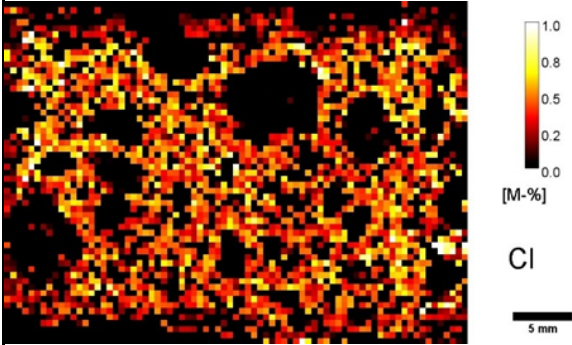
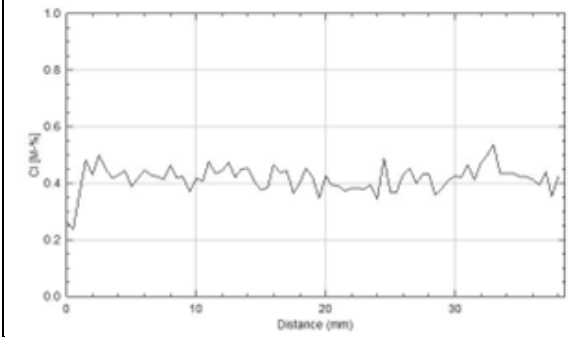
In contrast, the specimens incorporating CEM III show higher scattering with the chloride profiles than the specimens of CEM I with no significant difference between the chloride contents.

Table 23: LIBS Measurements (Chlorides) of the specimens produced with blast furnace cement

Specimen	LIBS Cl Measurements	
-	Photo	Chlorides
CEM III 1.5 %		 
CEM III 3.0 %		 

To further investigate the limited chloride redistribution observed in the CEM III specimen with 3.0 wt.-%/c, an additional LIBS measurement targeting chloride elements was conducted on the fractured surface of a prism-shaped specimen. The results are presented in Table 24.

Table 24: LIBS Measurements (Chlorides) of the prism specimen produced with blast furnace cement and 3.0 wt.-%/c. Chlorides in the RCA parent concretes

Specimen	LIBS Cl Measurements	
-	Photo	Chlorides
CEM III 3.0 % Prism		 

The data presented in Table 24 clearly indicate that, after six months a significant redistribution of chlorides from the RCAs into the cementitious matrix of the CEM III 3 % concrete specimen has taken place.

4.5 Investigation of the chloride redistribution in carbonated concrete

To investigate the potential impact of carbonation on chloride-containing RCAs, the aggregates were artificially carbonated in a carbonation chamber at a CO_2 concentration of 3 % until full carbonation was achieved. Subsequently, specimens were prepared following the procedure described in Section 4.4. Further LIBS measurements (Section 4.4) were then carried out to determine whether a difference exists between carbonated and non-carbonated RC-specimens with respect to the redistribution of chlorides which may correlate with potential corrosion activity.

For CEM I with 1.5 wt.-% Chlorides no significant difference in the chloride distribution can be observed for un-carbonated or carbonated specimens. But with higher chloride content (3.0 wt.-%) the chloride content analysed the LIBS is much lower for carbonated RCA. Also for CEM III it is found that the analysed chloride content for carbonated concrete is lower than for un-carbonated concrete with similar scattering.

Table 25: LIBS Measurements (Chlorides) of the specimen produced with Ordinary Portland Cement, not carbonated and carbonated

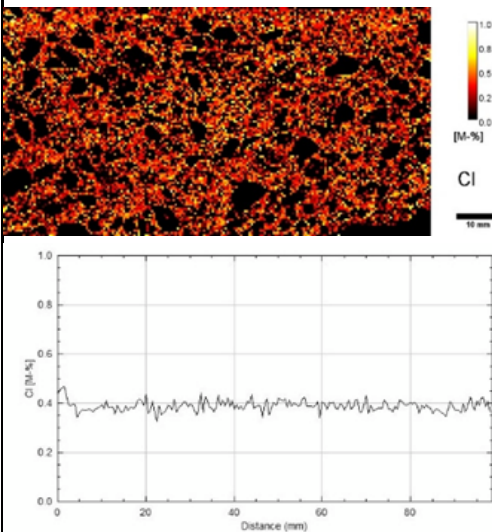
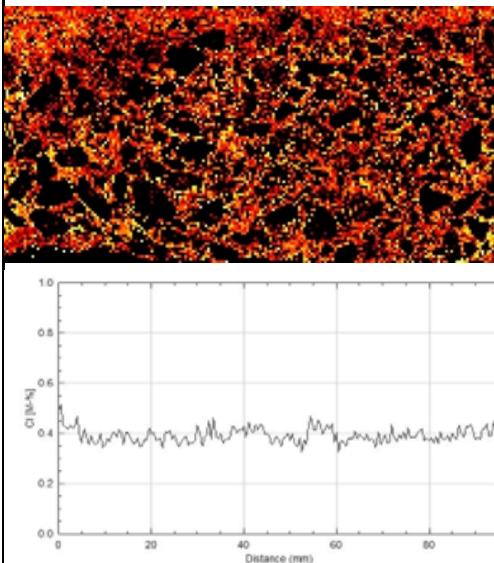
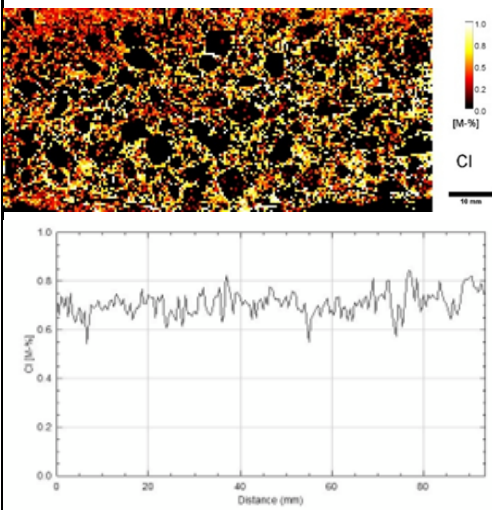
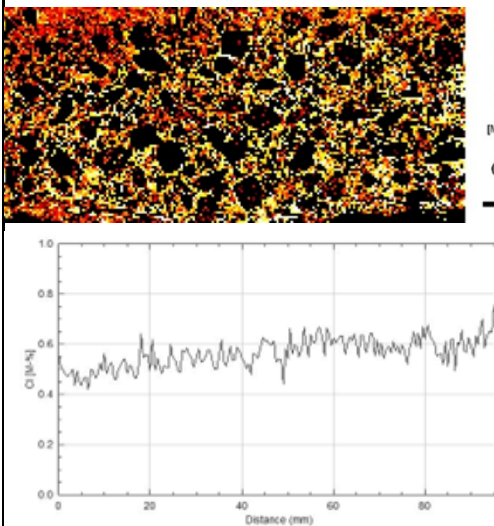
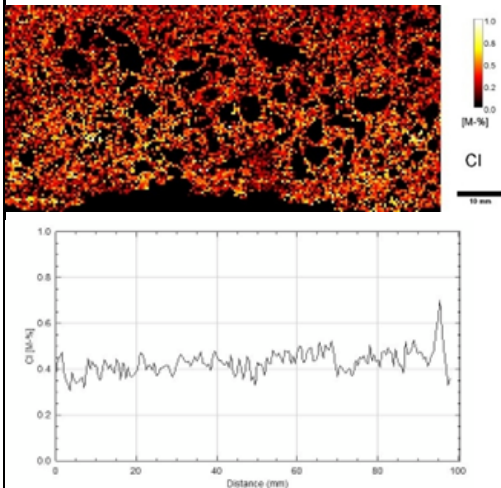
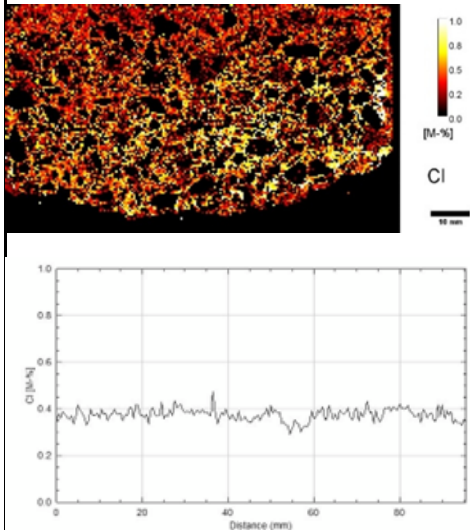
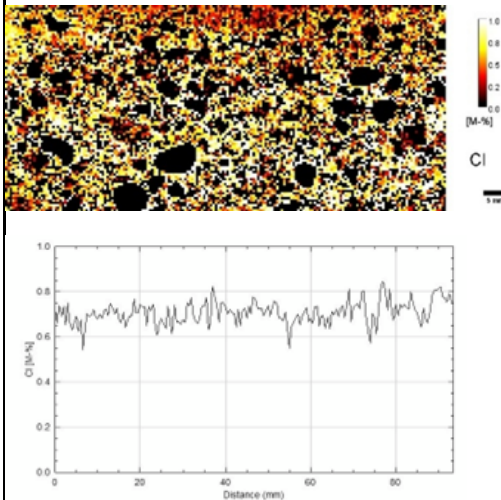
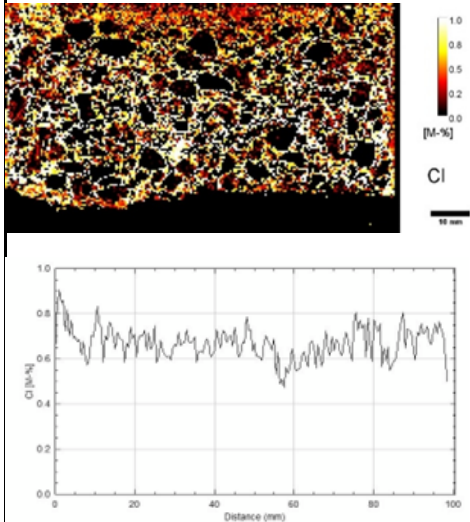
Specimen	LIBS Cl Measurements	
-	not carbonated	carbonated
CEM I 1.5 %		
CEM I 3.0 %		

Table 26: LIBS Measurements (Chlorides) of the specimen produced with Blast Furnace Cement, not carbonated and carbonated

Specimen	LIBS Cl Measurements	
-	not carbonated	carbonated
CEM III 1.5 %		
CEM III 3.0 %		

From the binding theory of chlorides in cement stone, it could be expected that chloride-containing, carbonated recycled concrete aggregate (RCA) would lead to increased leaching of chlorides. However, this could not be substantiated based on the LIBS investigations. While lower chloride contents are analytically detectable for many carbonated test specimens compared to non-carbonated ones, this may be due to a slightly increased leaching of the carbonated test specimens. However, the effect is not significant.

Overall, the LIBS investigations show that chloride exposure is very uniformly distributed in recycled concrete (RC) concrete. While variations can be observed, they do not lead to a reassessment of the corrosion risk.

5 CO₂-integration in recycled concretes

5.1 Analysis of typical recycled concrete aggregates

Recycled aggregates investigated in Brazil

The description of how the recycled aggregated was obtained, as well as the characterization of the fine and coarse fractions, are found in Section 3.6. of this report.

5.2 Natural Carbonation behaviour of recycled concrete aggregates

5.2.1 "Natural" carbonation of recycled aggregates in Brazil

All carbonation of the recycled aggregates were carried out in an environmental chamber to maintain control of the variables involved. The natural-like carbonation environment was set in the chamber by using a fixed $30 \pm 2^\circ\text{C}$, relative humidity of $65 \pm 5\%$ and a CO₂ concentration of 0.04 %. The process was applied for 7 days. The results of the "natural" carbonated recycled aggregates is shown together with results of forced carbonated samples in the following Sections. They are identified as "Atm".

5.2.2 " Natural" carbonation of recycled aggregates in Austria

The recycled aggregates described under 3.6.2 were either obtained from recycling companies or taken directly from the demolished building, as described above. In both cases a very low degree of carbonation can be seen if a phenolphthalein test is applied. Figure 44 shows the typical storage of processed and certified recycled aggregates in piles exposed to environmental influences.



Figure 44: Processing and storage of concrete recyclate in landfills (background), typical degree of carbonation of the aggregates (bottom left), advanced carbonation (bottom right)

Individual grains of rock from the processing were taken from different depths of the deposit and subjected to a phenolphthalein test. Where grains from the surface of the deposit show partly full carbonation of the surface with minor carbonation depth, those samples which were taken from greater depth or covering, poor carbonation can be seen. It can be assumed that the lack of aeration during storage is not sufficient to allow carbonation with atmospheric CO_2 in the medium term. With reference to the measurements shown in Section 5.3.1, the carbonation progress at low CO_2 concentrations is very slow progressing after surface saturation has been completed. Due to the storage of concrete recyclates with the same recyclate quality but different origins and compositions, there is a wide range of carbonation levels in the material mixture.

5.3 Forced Carbonation of recycled concrete aggregates

5.3.1 Accelerated carbonation of recycled aggregates in Brazil

Accelerated carbonation was carried out for 7 days in an environmental chamber under a CO_2 concentration of 5 %, $30 \pm 2^\circ\text{C}$ and relative humidity of 65 ± 5 %. Additionally, RCA were tested at three different moisture contents (MC), 0 %, 30 %, and 50 %, to evaluate how varying humidity conditions would affect the carbonation process. These samples were nominated Acc 0, Acc 30 and Acc 50.

Establishing the CO_2 uptake by weight gain

The first method to quantify the CO_2 captured by the fine and coarse recycled aggregates was by establishing the mass gain during the process, as shown in Figure 45 and Table 27 below.

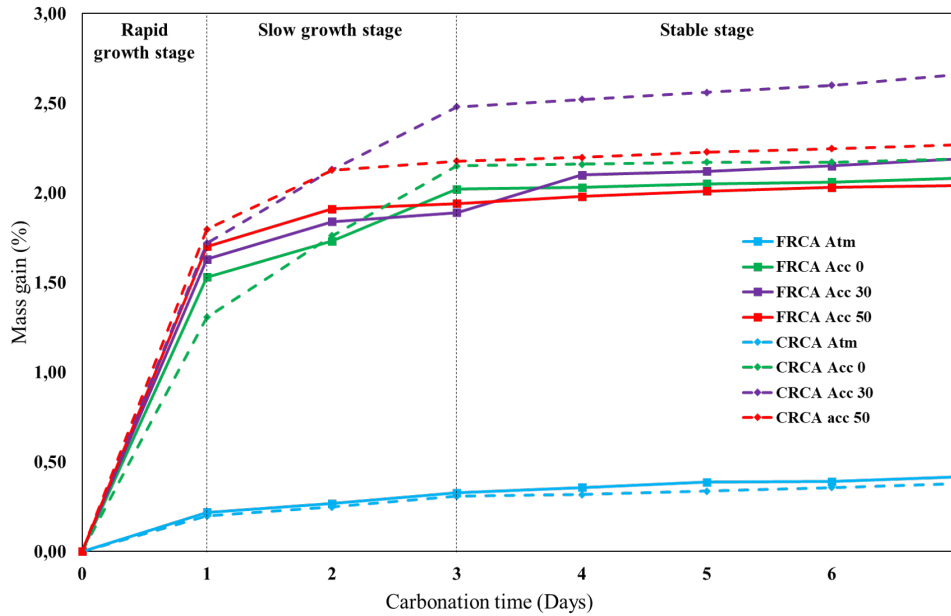


Figure 45: Values for mass gain of the samples after 7 days of accelerated carbonation.

Table 27: Mass gain of fine and coarse recycled aggregates under different conditions

Aggregate type	Atm (%)	Acc 0 (%)	Acc 30 (%)	Acc 50 (%)
FRCA	0.42	2.08	2.19	2.04
CRCA	0.38	2.19	2.66	2.27

The CO₂ uptake potential using the mass gain method was calculated by

$$RCA_{CO_2 \text{ uptake}} (\%) = \frac{(M_{\text{carbonated}} - M_{\text{water}})(\%) \times OM_{\text{content}} (\%)}{100}$$

Where,

$RCA_{CO_2 \text{ uptake}}$ = CO₂ uptake potential for RCA.

$M_{\text{carbonated}}$ = Mass of the sample after the accelerated carbonation process.

M_{water} = Mass of water present in the sample.

OM_{content} = Amount of old mortar attached to RCA surface.

Considering the Atm condition (0.42 % for FRCA and 0.38 % for CRCA) as the water gain (M_{water}) and using the old mortar (OM) values (30.99 % for FRCA and 28.24 % for CRCA), the results in Table 28 indicated that for each kilogram of RCA, the CO₂ content that could be captured ranges from 5.02 to 5.49 g CO₂ for each kg of FRCA and from 5.11 to 6.34 g CO₂ for a kg of CRCA.

Table 28: CO₂ uptake potential by mass gain method.

Aggregate type	CO ₂ uptake (%)	CO ₂ (g/kg RCA)	Kg CO ₂ /ton RCA
FRCA Acc 0	0.514	5.14	5.14
FRCA Acc 30	0.549	5.49	5.49
FRCA Acc 50	0.402	5.02	5.02
CRCA Acc 0	0.511	5.11	5.11
CRCA Acc 30	0.644	6.44	6.44
CRCA Acc 50	0.534	5.34	5.34

Determining the CO₂ uptake by TGA/DTG analysis

The TGA/DTG curves for the recycled aggregates in the 'As-received' condition (prior to carbonation) and after accelerated carbonation under various conditions are shown in the Figure below. All TGA curves exhibited similar behavior: a slight mass loss between 0°C and 100°C, attributed to water evaporation, followed by a gradual decrease in mass from 100°C to 650°C, which corresponds to the decomposition of ettringite, C-S-H, and less crystalline phases of calcium carbonate. A substantial mass loss was observed between 650°C and 950°C. The DTG curves indicate that the mass loss between 600°C and 900°C is related to the decomposition of CaCO₃. The curves for the 'As-received' condition exhibited lower mass loss and DTG peak values compared to the other conditions, indicating that exposing the RCA to the climatic chamber effectively promoted carbonation. The highest mass losses and corresponding peaks in the DTG curves followed the order: 'Atm', 'Acc 0', 'Acc 50', and 'Acc 30', correlating with the degree of carbonation and CO₂ uptake. Furthermore, when comparing granulometry, the coarse fraction exhibited higher mass loss than the fine fraction, confirming the results from the mass gain method.

TG/DTG curves of carbonated and non-carbonated fine (left) and coarse (right) aggregates

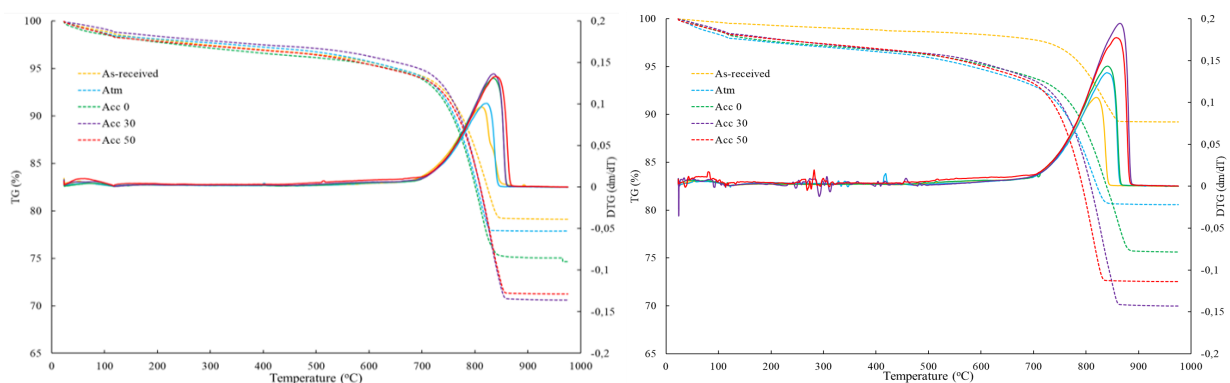


Figure 46: TG/DTG curves of carbonated and non-carbonated fine (left) and coarse (right) aggregates

Additionally, the CO₂ captured and the total CO₂ content after accelerated carbonation under various conditions are summarized in the Table below. Notably, after the 'As-received' measurements, the fine RCA fraction captured less CO₂ than the coarse fraction, confirming the mass gain results, where CRCA showed more carbonation than FRCA after 7 days. Furthermore, the total CO₂ captured by coarse recycled aggregates was consistently higher across all conditions compared to the fine fraction. When considering water content, both FRCA and CRCA in the 'Acc 30' condition showed the highest carbonation results, supporting the mass gain method findings after 7 days of exposure and emphasizing that oversaturation of samples can hinder carbonation reactions. The results revealed that for each kilogram of RCA, the CO₂ content that could be captured ranged from 4.99 to 43.16 g CO₂ for FRCA and from 2.53 to 62.53 g CO₂ for CRCA, which corresponds to 4.99 to 43.16 kg of CO₂ per ton of FRCA and 2.53 to 62.53 kg of CO₂ per ton of CRCA.

Table 29: CO₂ uptake potential through TGA/DTG

Aggregate	CO ₂ uptake (%)	Total CO ₂ (g/kg RCA)	Kg CO ₂ /ton RCA
FRCA as-received	4.99	4.99	4.99
FRCA Atm	1.59	9.92	9.92
FRCA Acc 0	5.02	20.54	20.54
FRCA Acc 30	12.32	43.16	43.16
FRCA Acc 50	10.23	36.70	36.70
CRCA as-received	2.53	2.53	2.53
CRCA Atm	13.63	17.84	17.84
CRCA Acc 0	12.72	38.44	38.44
CRCA Acc 30	21.25	62.53	62.53
CRCA Acc 50	16.81	49.99	49.99

5.3.2 Accelerated carbonation of recycled aggregates at FHCW

The accelerated carbonation tests were carried out for over several days (varying test durations were defined depending on the behaviour of the tested sample; 1 – 10 days) in an environmental chamber under a CO₂ concentration of 20 – 30 %, 22 ± 2 °C and relative humidity of 60 - 80 %.

A specially developed test setup was used to carry out the carbonation tests, which allows the CO₂ uptake to be monitored in real time. A sketch of the test setup is shown in Figure 47. The gas-tight test chamber does not have to be opened for this purpose, but can remain closed throughout the duration of the test. The CO₂ uptake is calculated via the CO₂ consumption and the associated weight loss of the CO₂ gas tank. At the end of the test, by determining the dry mass of the sample and comparing it with the dry mass at the start of the

test, the determined value can be compared with the measurement during the test for validation. The sample is static in the carbonation chamber for the entire duration of the test and is flowed through from below on one side.

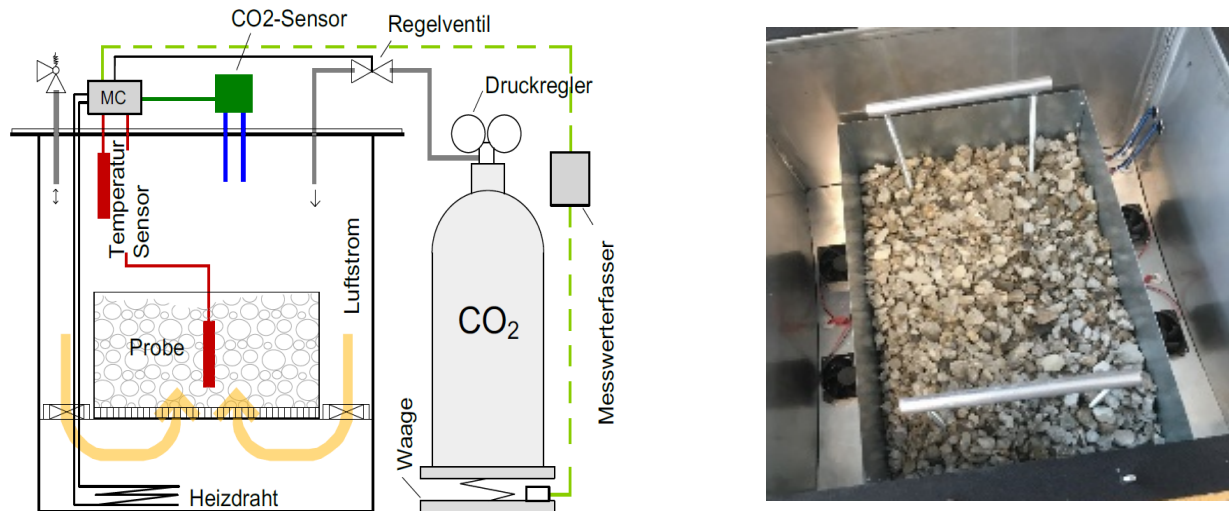


Figure 47: Carbonation test setup for real-time monitoring the CO₂ consumption of aggregates or concrete samples (left), recycled aggregate inside the test chamber (right)

Initial tests on the suitability of the apparatus were carried out under controlled humidity conditions and with hydrated lime, as the absorption capacity can be particularly easily demonstrated theoretically on the basis of the chemical reaction and compared with the practical data.

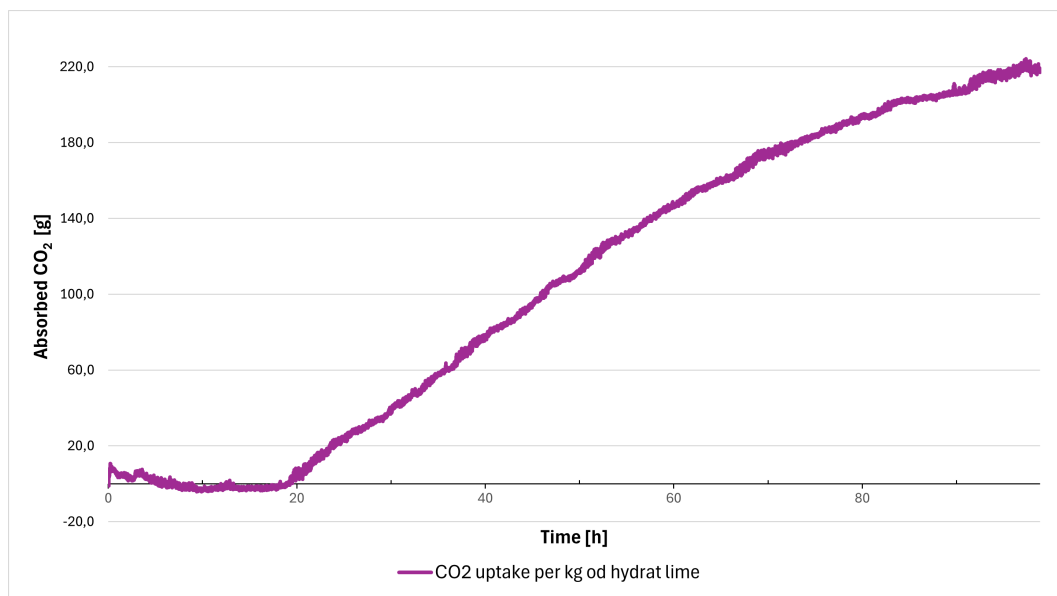


Figure 48: CO₂ uptake of hydrated lime over 100 h plotted per kg paste

For calibration purposes, pure hydrated lime is used, as its high hydroxide content enables a rapid carbonation reaction. White hydrated lime (CL 90-S) serves as the base material, mixed with water at a ratio of 1:1. A total of 2 kg of this paste is prepared and poured into a mold measuring 38 cm × 15 cm. During hardening in the reactor atmosphere, the moisture generated by the carbonation reaction raises the relative humidity above the dew point temperature. This leads to the formation of a thin water film on the surface of the sample, which hinders CO₂ diffusion and slows down the carbonation process. To solve this issue, a drying agent is introduced, allowing the humidity to be kept below the dew point. This ensures a continuous CO₂ flow and maintains full reactivity of the sample. The CO₂ uptake follows a characteristic pattern: after an initial delay—caused by the time needed for the internal climate of the reactor to stabilize—the carbonation process begins, leading to an exponential increase in CO₂ uptake, which gradually levels off. In total, approximately 220 g of CO₂ are absorbed per kilogram of binder.

Carbonation tests were carried out on all recycling aggregates described in 3.6.2. All test results show the CO₂ uptake in M-% of the total mass of the dry sample, unless otherwise stated. It must therefore be emphasized that carbon dioxide can only be bound in reactive material that is only contained in the binder or cement. The absorption capacity of a concrete recycle is therefore strongly dependent on the binder content of the demolition concrete. In addition, only the residual absorption capacity of the recycle can be determined as part of the forced carbonation test, as carbonation takes place slowly but continuously under natural storage conditions in the open air. This means that all samples from larger recycle storage facilities whose storage period at the recycling center cannot be precisely determined are to be considered pre-carbonated to a certain degree, see Sections 5.2.1 and 5.2.2.

For this reason, separate concrete samples with a standard mix design were produced and crushed immediately before the carbonation process. In this way, a reference for hydrated but largely uncarbonated concrete was to be created. The samples were crushed in a jaw crusher to a maximum grain size of 22 mm and then separated into 2 grain fractions:

- 0 - 8 mm and
- 8 – 22 mm.

The tests for CO₂ absorption with this method were carried out separately on the grain fractions 0 - 8 mm and 8 - 22 mm in order to be able to demonstrate a different behavior of fine and coarse aggregates. The recorded data of both sample types was analyzed after a carbon dioxide exposure of 24 hours.

The reaction conditions were kept constant over the entire duration of the experiment. The samples were heated to 40 °C, the CO₂ content in the sample chamber was set at 20 % and the ambient humidity was kept at 70 %.

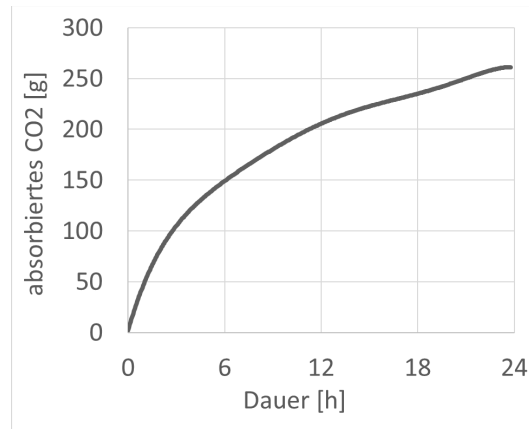


Figure 49: Reference carbonation curve of recycled concrete aggregate 8 - 22mm crushed immediately before the test

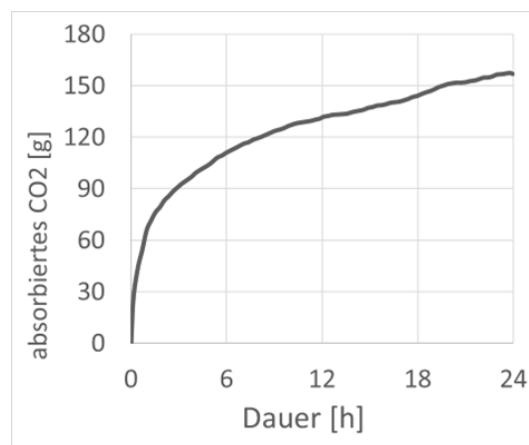


Figure 50: Reference carbonation curve of recycled concrete aggregate 0 – 8 mm crushed immediately before the test

The top two diagrams, Figure 49 and Figure 50, show the absorption potential of uncarbonated crushed concrete over an exposure of 24 hours.

Figure 51 shows a comparison of the CO₂ uptake of both samples over 24 h. It is noticeable that the fine grains, which generally contain a higher proportion of cement and also has a larger surface area, does not show any greater storage capacity. This is due to the fact that the sample is too densely packed in the sample container due to its fineness and therefore no ideal air flow is achieved.

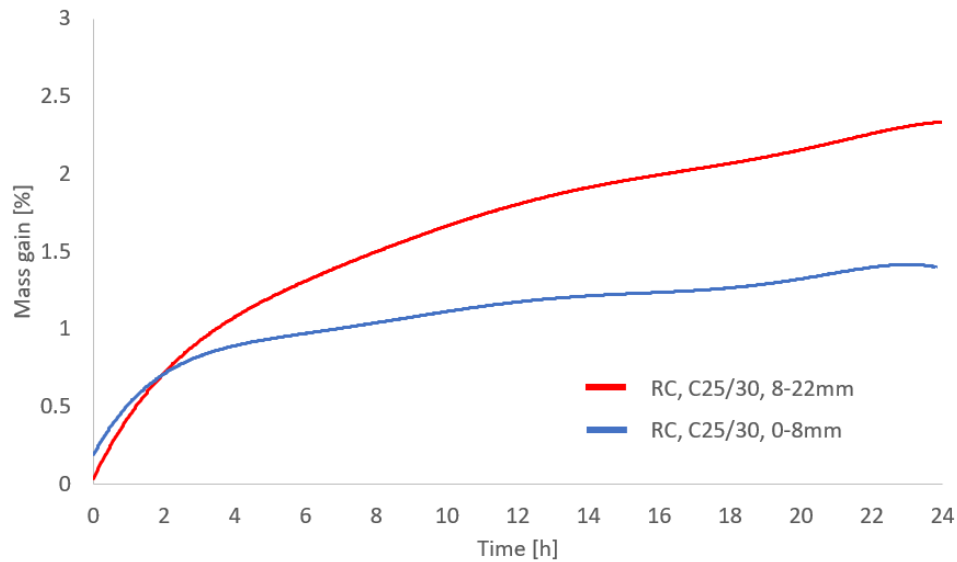


Figure 51: Comparison data recording of CO₂ uptake under accelerated conditions as described in 5.3.2

The fine grain initially shows very rapid CO₂ absorption, which can be explained by the high surface area to mass ratio and the high cement content. Due to the incomplete saturation of the fine grain, the test ends with a CO₂ uptake of around 1.42 m-% of the sample. The coarse grain with 8 - 22 mm shows an initially more moderate saturation progression with increasing flattening from 2 h after the start of the CO₂ exposure. CO₂ uptake is not completed after 24 h in this case. By using phenolphthalein tests on split grains no significant remaining storage potentials can be seen. Testing coarse grain sizes of 8 - 22mm leads to a CO₂ absorption potential of 2.35 m-%.

Influence of CO₂ Concentration, Temperature, and Sample Moisture

The following Section investigates the influence of CO₂ concentration, temperature, and sample moisture on the carbonation behavior of concrete. For this purpose, concrete cubes are exposed to forced carbonation in a CO₂ reactor under varying conditions. The following test setups are applied:

- Test 1: 25 °C and 20 % CO₂ concentration
- Test 2: 25 °C and 30 % CO₂ concentration
- Test 3: 50 °C and 20 % CO₂ concentration
- Test 4: 50 °C and 30 % CO₂ concentration
- Test 5: 50 °C and 30 % CO₂ concentration with prior drying of the samples at 110 °C for 24 hours

The results of the individual tests are shown below. Each data point represents the absolute CO₂ uptake measured at a specific time point in 15-minute intervals. A regression analysis

was performed using a square root function to model the progression of CO₂ uptake over time, shown as the solid line.

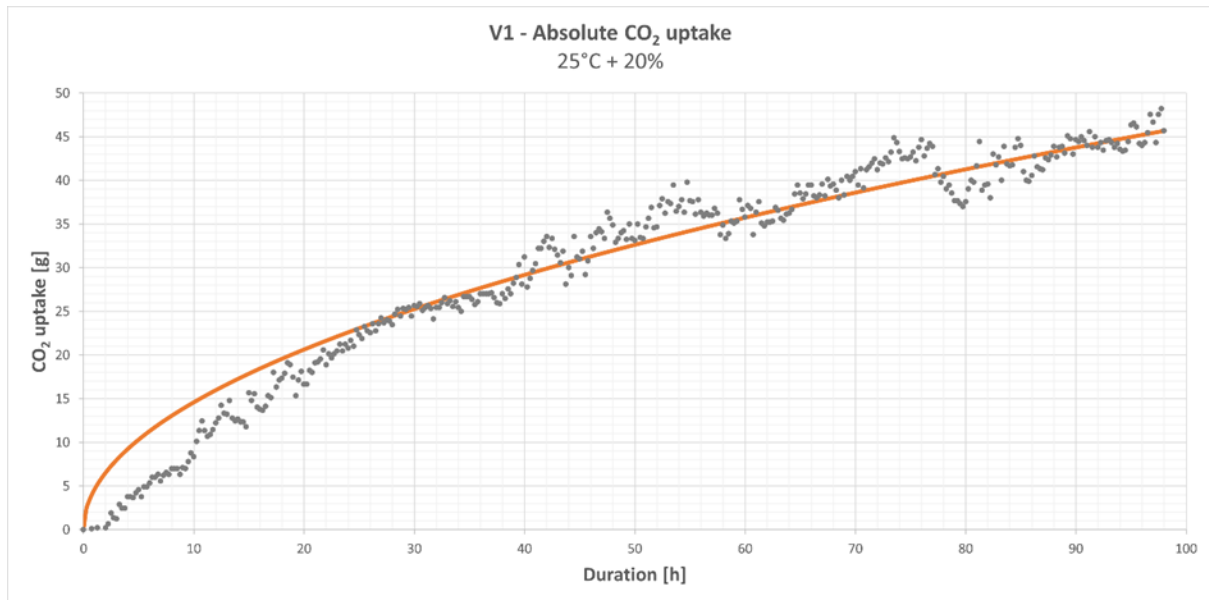


Figure 52: Evaluation diagram of test 1: 25 °C and 20 % CO₂ concentration, absolute CO₂ uptake: 52,7 g

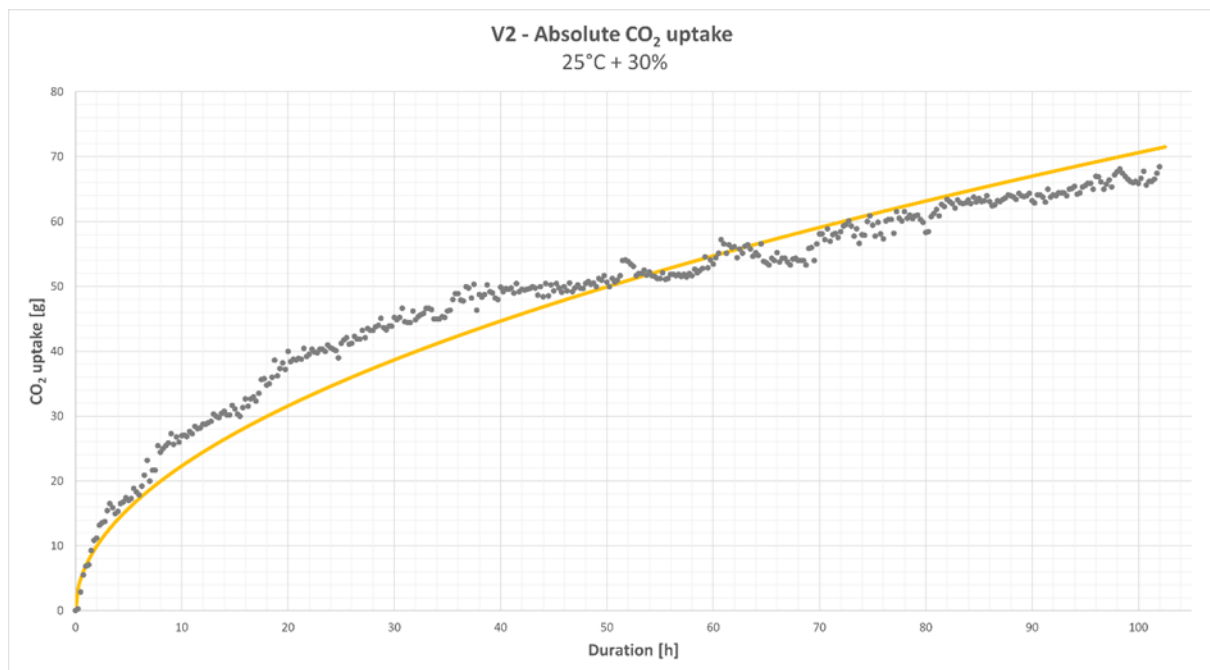


Figure 53: Evaluation diagram of Test 2: 25 °C and 30 % CO₂ concentration, absolute CO₂ uptake: 70,1 g

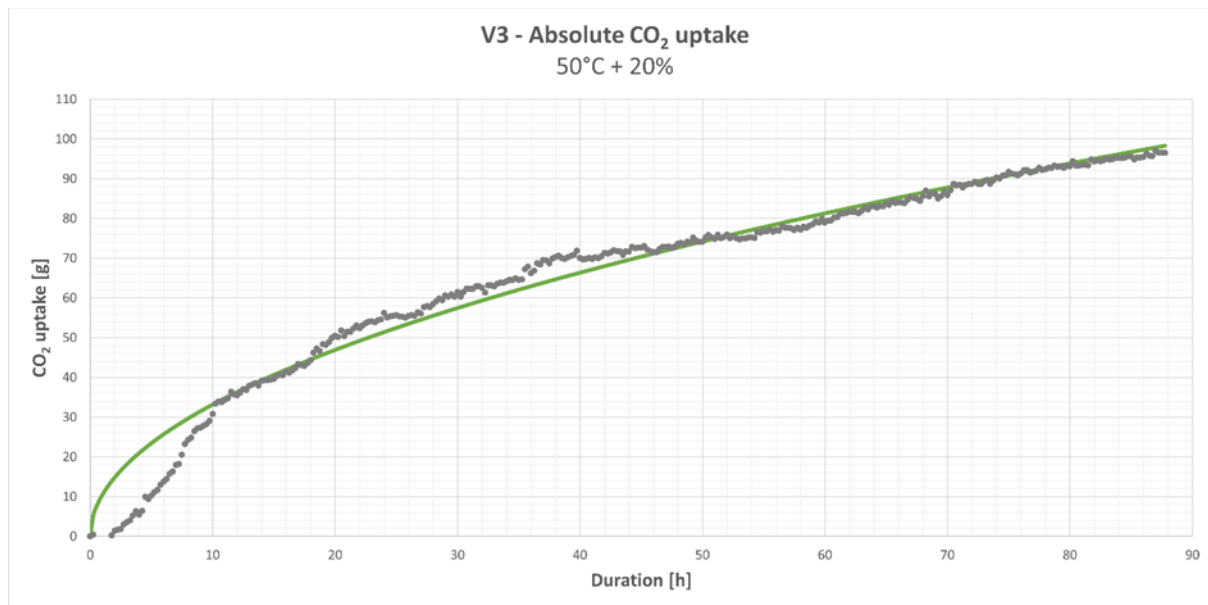


Figure 54: Evaluation diagram of test 3: 50 °C and 20 % CO₂ concentration, absolute CO₂ uptake: 99,7 g

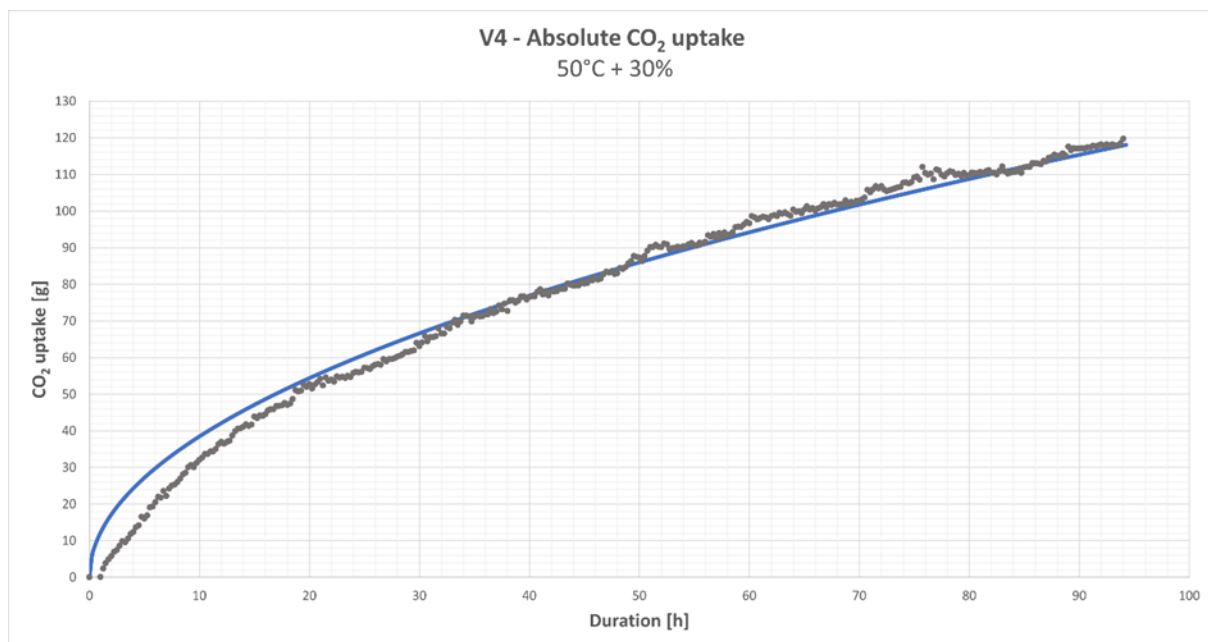


Figure 55: Evaluation diagram of test 4: 50 °C and 30 % CO₂ concentration, absolute CO₂ uptake: 123,7 g

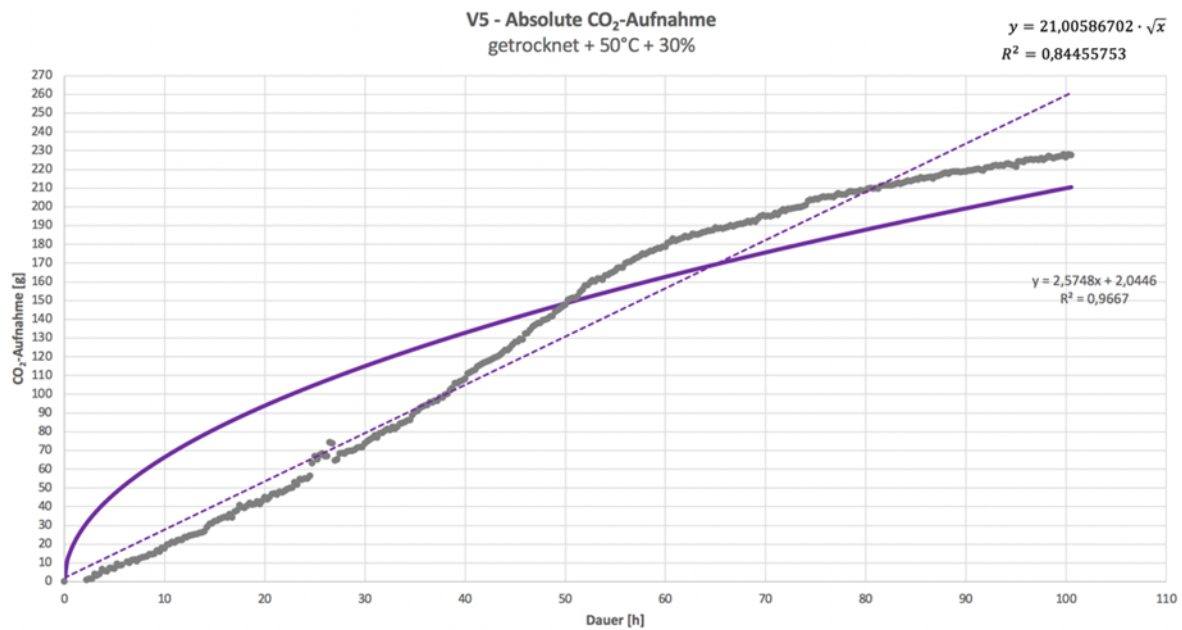


Figure 56: Evaluation diagram of test 5: 50 °C and 30 % CO₂ concentration with prior drying of the samples at 110 °C for 24 hours, absolute CO₂ uptake: 178,6 g

The following graph presents and compares the regression curves of all conducted tests on cube specimens.

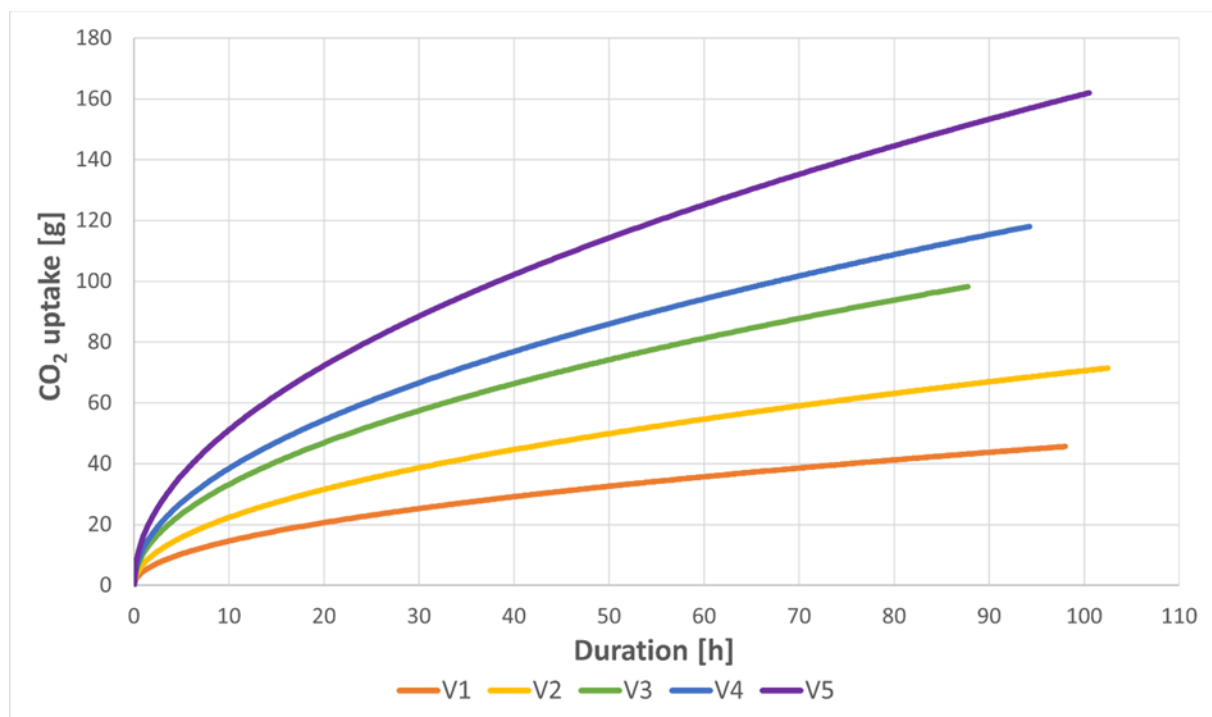
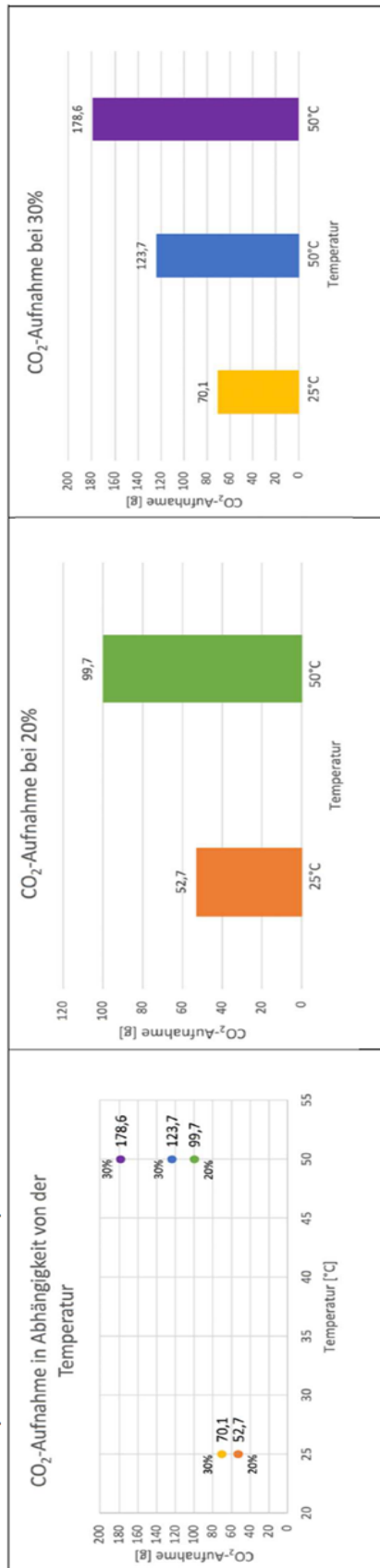


Figure 57: Comparison of the absolute CO₂ uptake of samples V1 - V5

The lowest CO₂ uptake was measured in the first experiment, Figure 52, (25 °C, 20 % CO₂ concentration) at 52.7 g. In the second experiment, Figure 53, (25 °C, 30 % CO₂ concentration), the test cube absorbed 70.1 g of CO₂. This illustrates that a higher CO₂ content has a positive effect on carbonation. A similar trend was observed in the experiments conducted at 50 °C. While the test cube in the third experiment, Figure 54, at 20 % CO₂ concentration, absorbed 99.7 g of CO₂, it absorbed 123.7 g in the fourth experiment, Figure 55, at 30 % CO₂ concentration. Again, it was found that a higher CO₂ concentration can optimize CO₂ storage. At the same time, a comparison with the 25 °C experiments shows that higher temperatures can further accelerate and intensify CO₂ uptake. The highest CO₂ uptake was recorded in the fifth experiment, Figure 56. The test cubes were dried at 110 °C before the carbonation process and then subjected to the same conditions as in experiment 4, i.e., at 50 °C and 30 % CO₂ concentration. One concrete cube absorbed 178.6 g of CO₂. This clearly shows that pre-drying the test cubes has a significant positive effect on CO₂ storage capacity. However, the key to maximizing CO₂ storage capacity is not only that the sample becomes more porous through drying, but also that sufficient moisture is available during the carbonation process.

In addition, the carbonation depths of the test cubes were determined, which support these findings. For example, the test cube from the first experiment also showed the lowest carbonation depth, measured at 5 mm, while a maximum penetration depth of 35 mm was found in the fifth experiment.

Effect of temperature on CO₂ uptake



Effect of CO₂ concentration on CO₂ uptake

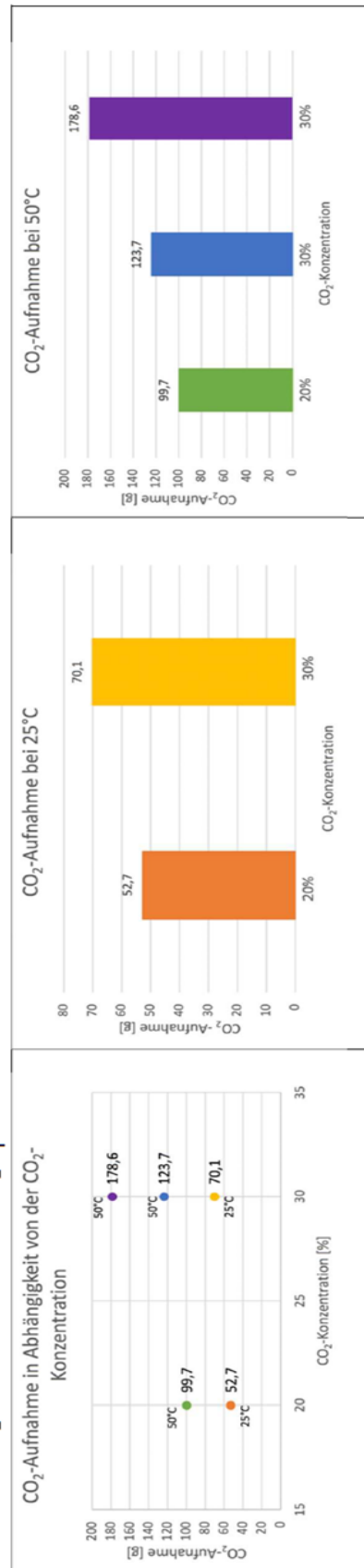


Figure 58: Summary of effects of temperature (left) and CO₂ concentration (right) on the CO₂ uptake of cubic specimens

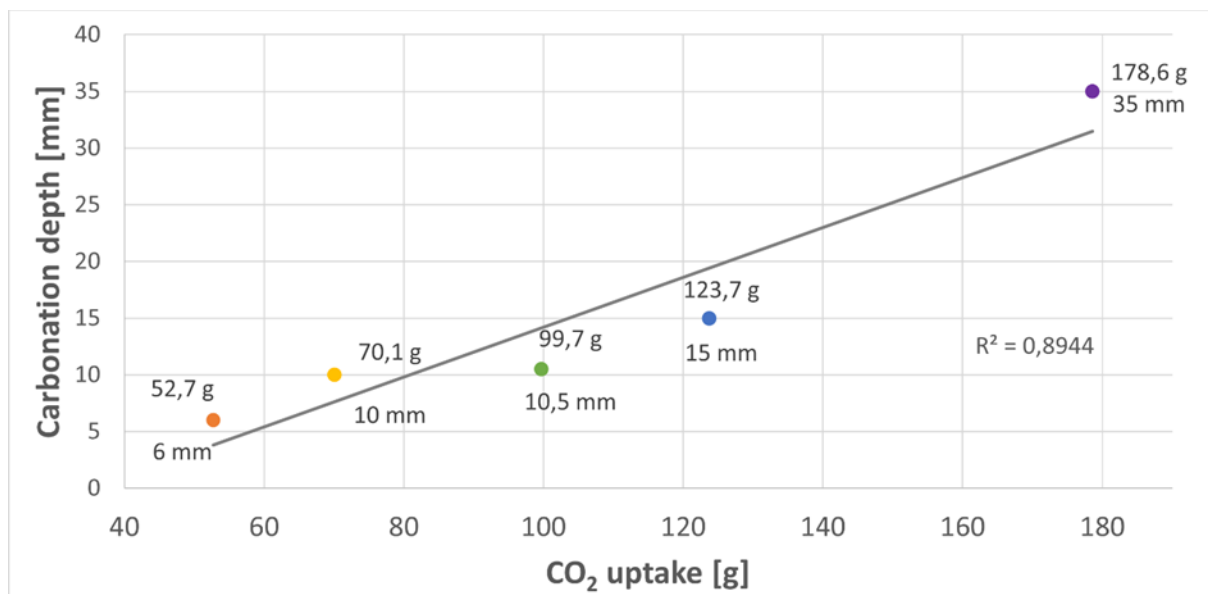


Figure 59: Correlation between carbonation depth and absorbed CO₂

In this scatter plot, the amount of absorbed CO₂ in grams is plotted on the x-axis, and the carbonation depth in millimetres is plotted on the y-axis. The individual points represent the respective experiments and are color-coded. To illustrate the correlation, a linear trendline has also been added. A coefficient of determination of 0.894 indicates a very strong linear relationship between CO₂ uptake and carbonation depth, Figure 59. It can be seen that the more CO₂ the sample cubes absorbed, the deeper the carbonation front progressed within the material. This clear correlation confirms the previously observed individual results from the series of experiments and highlights the close relationship between CO₂ uptake and the advancement of the carbonation reaction in the concrete

In summary, it can be stated that leaving the sample static in the carbonation chamber results in poor air flow through the aggregate in the case of fine grains and therefore incomplete CO₂ enrichment of the sample. This can be observed through all tests conducted and should be considered for any industrial forced carbonation process or machinery design.

Following the carbonation of the recycled aggregates, chloride measurements were carried out again using the method described above in order to analyse any differences. However, it must be emphasized in this case that chloride content tests are only carried out on a small sample quantity, which is why there can be a large scatter in the results. It is therefore advisable to test a larger number of samples per test series in order to compensate for possible fluctuations in the chloride content due to different exposures of the demolition concrete. This fact can be considered particularly important, as the determination of the chlorine content before and after carbonation cannot be carried out on one and the same sample.

Table 30: Overview of the chloride content of the recycled material samples in the fully carbonated state

Sample	Carbonation state	Chloride content
RB II, 0/63, U7, U-A	Full carbonation	<0.12 M-%
RB III, 0/32, U-A	Full carbonation	<0.05 M-%
RMH III, 0/63, U-A	Full carbonation	<0.09 M-%

The comparison of the measurable chloride contents shows an increase in 2 out of 3 samples, which is possibly due to the additional release of chloride ions through the storage of CO₂ in the cement paste. The results thus agree with approaches from the literature, but should be tested in greater depth with a larger number of samples, especially with regard to their effects on material reuse for the production of recycled concretes.

Table 31: Comparison of the measurable chloride content of the recycle sample before and after forced carbonation

Sample	Chloride content, before carbonation	Chloride content, after carbonation
RB II, 0/63, U7, U-A	<0.05 M-%	<0.12 M-%
RB III, 0/32, U-A	<0.05 M-%	<0.05 M-%
RMH III, 0/63, U-A	<0.05 M-%	<0.09 M-%

5.4 Conclusions for designing of an industrial carbonation process

The development and further development of a specific testing equipment for the carbonation process of concrete recycles forms the basis for the considerations regarding the industrialization of the described process. Basic technical requirements can be specified from the test setup described and on the basis of research into international reference projects.

One of the most important findings from the preliminary tests was the avoidance of a stationary carbonation process, as the static storage of the sample material has advantages in terms of the energy input required to rearrange the material to be charged but is also detrimental to

the enrichment of the cement stone with CO₂. Particularly fine-grained recyclates are naturally problematic in terms of their flowability due to their higher packing density. At the same time, the fine fraction of concrete recyclates is particularly rich in reactive cement paste (see WP 1) and therefore particularly well suited to permanently binding CO₂.

A further principle from a process engineering perspective is that cyclical processes should be avoided in order to create a continuous and therefore more industrializable process. To this end, it is recommended that an additional process step be added to existing plants in the cement and concrete industry. Here, for example, the treatment of concrete recyclates can be combined with waste gases from cement production. A particular suitability exists here due to the residual heat and above all due to the good waste gas purification of the rotary kilns with a high proportion of CO₂ in the gaseous emissions.

The direct integration of process waste gases is therefore obvious. However, even the decentralized treatment of demolition concrete has great ecological advantages even taking industrial processing and related energy consumption into account, Figure 60 (acc. to (Tiefenthaler, et al. 2021)).

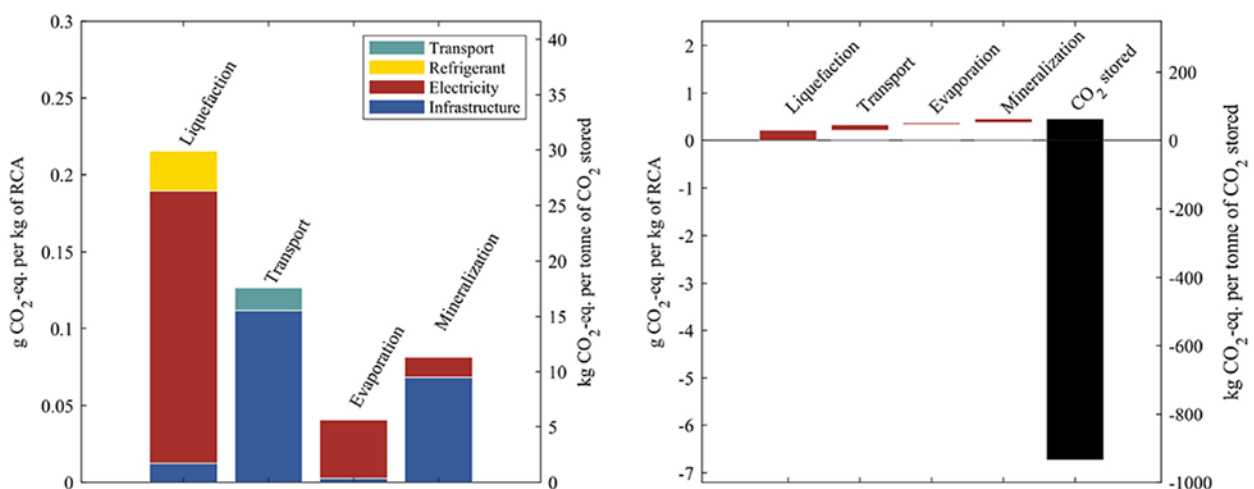


Figure 60: The GWP of the sub-processes of the negative emission value chain are visualized along four main process categories. (Right) The cumulative GHG emission of the value chain, including the negative emissions of the mineralization plant (Tiefenthaler, et al. 2021)

An overview life cycle assessment, also in comparison with international scientific references, illustrates the ecological advantage of using old concrete as a CO₂ sink. According to the investigations in WP3, the basically natural process is much slower compared to accelerated carbonation and can therefore only be integrated into economic processes with technical possibilities. However, additional process engineering should be avoided as far as possible in order to keep the resulting additional expenditure as low as possible and not to reduce the ecological advantage resulting from the CO₂ storage capacity of the waste concrete to an excessive extent.

In accordance with the investigations in WP 3, (Section 5.4) and reference studies, Figure 61, the main focus is to be placed on the CO₂ enrichability and usability of fine aggregates, as there is great storage potential. This is due to the fact that the reactive cement paste content is higher for fine grain fractions after processing than for coarser grain groups, where significantly more natural aggregates are present by mass in relation to the adhering cement paste, see Section 3.7 and Figure 50.

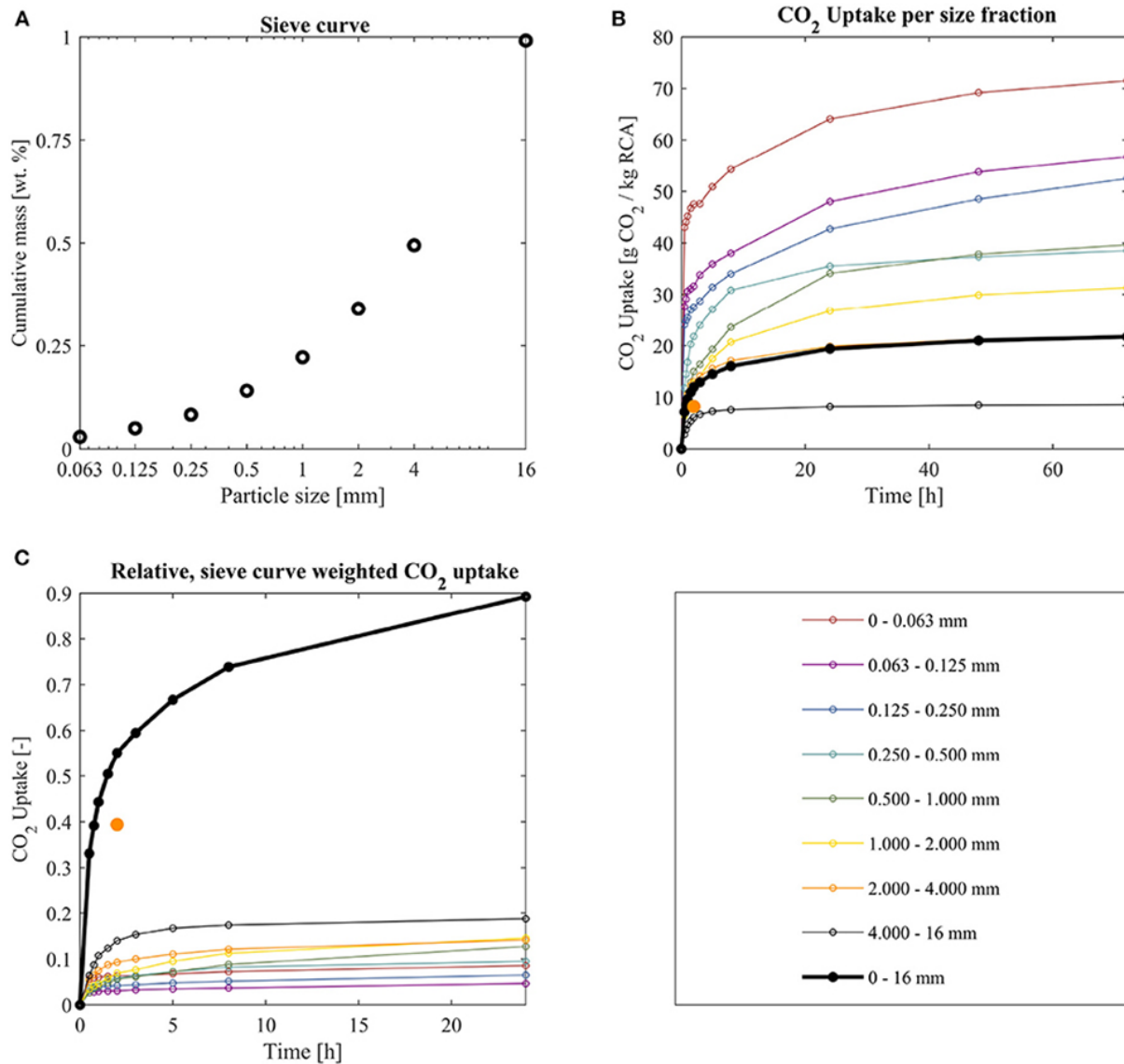


Figure 61: (A) The sieve curve is plotted over the particle size. (B) The CO₂ uptake of different particle size fractions is plotted over the experimental time. (C) The CO₂ uptake relative to the 72 h uptake is plotted over time. Furthermore, the relative contribution of the different size fraction to the total uptake is visualized. The orange dot indicates the current operating point in the industrial plant. (TIEFENTHALER et al., 2021)

6 Development of refined hygrothermal material models for concrete, determination of material properties and corrosion maps for the new concrete types

In this work package, material data for concrete was developed for the use in hygrothermal component simulation with WUFI. WUFI is a menu-driven PC program for the simulation of heat and moisture transport in building components. It was developed at the Fraunhofer Institute for Building Physics and is based on the most recent findings on the coupled temperature and moisture storage and transport in building materials. It can be used in the examination of multi-layered building components to determine for example whether the building moisture dries out or whether the building component moistens due to rain water absorption or vapor diffusion transport from the indoor climate. In the context of this project new material models for concrete are developed. Therefore, concrete formulations with Portland cement (CEM I) and blast furnace cement (CEM III) were investigated. These were chosen as they are the two most used binder groups. Since the focus of the project is on the investigation of corrosion in reinforced concrete and this can be significantly increased by any Cl ions contained, also samples with Cl ions (recycling concrete) with a mass-related chloride content of 1.5 %, 2 % and 3 % were also considered. All test specimens were formulated and produced within the project in the HSM lab. The salt content was added to the concrete mix during the mixing phase.

6.1 Refinement of the concrete specific material models

The work is split into two main fields. One aim of the project is to improve the concrete material model to better be able to simulate the hygrothermal performance, the limited liquid transport in the core area of exposed concrete. The second topic is to realistically predict potential corrosion progress of the reinforcement depending on the corrosivity of the concrete depending on carbonation and chloride content and the specific temperature and humidity conditions at the position of the steel.

Earlier studies (Holm, Krus und Künzle 1999) have shown that capillary water transport in concrete can be significantly reduced (over time) under natural weathering conditions. The authors cite various possible causes for the latter effect, whereby some permanent swelling and blocking of the capillary pores and the resulting prevention of capillary moisture transport are considered by the authors to be the most likely cause. This behaviour of freely weathered concrete and the resulting difference between measurement and calculation, which cannot directly consider this effect, is taken from (Holm, Krus und Künzle 1999) and shown below in Figure 62 as an example.

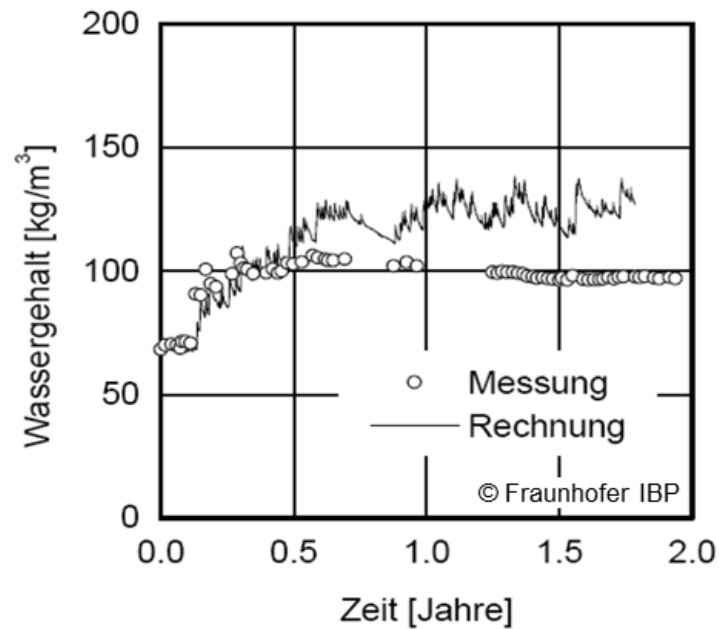
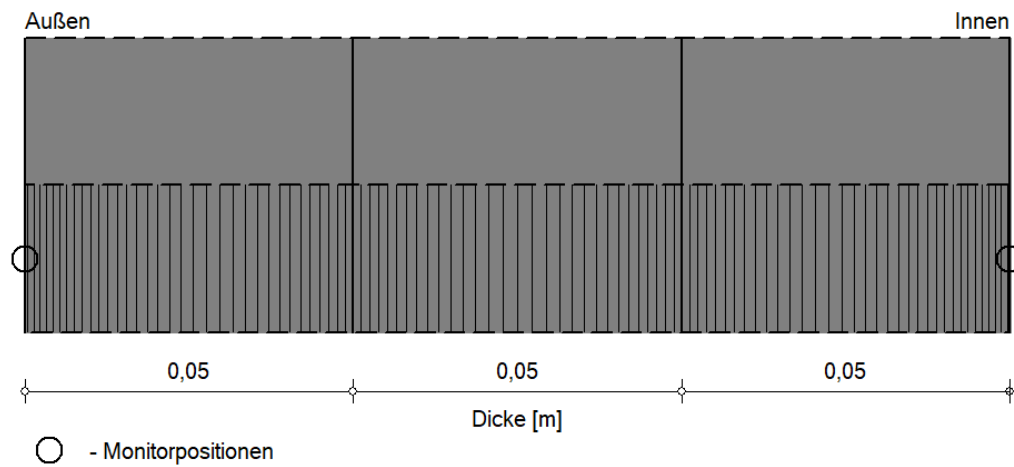


Figure 62: Comparison of the courses of the water content between hygrothermal simulation and measurement with an NMR-Scanner from (Holm, Krus und Künzel 1999).

The analysis of the moisture profiles in concrete (Holm, Krus und Künzel 1999) have shown that continuous liquid moisture transport only occurs up to a depth of approximately 5 cm. Therefore, in the first step, a tripartite structure for concrete is examined and compared with current field measurements. In this process, edge areas, each 5 cm thick, are combined with a central area without liquid water transport. The material is incorporated into the hygrothermal component simulation with three layers, where both outer layers are identical. This is illustrated for an exemplary component layer in WUFI® in Figure 63 below.



Materialien:

	- *Außenschicht_E_CEM III 0 % CI Flüssigtransport	0,05 m
	- *Kernschicht_E_CEM III 0 % CI kein Flüssigtransport	0,05 m
	- *Außenschicht_E_CEM III 0 % CI Flüssigtransport (Kopie)	0,05 m

Figure 63: Exemplarily cross-section of a concrete specimen for the hygrothermal component simulation with a thickness of 15 cm separated into three parts.

The fundamental influence is illustrated in the following comparison of the overall water content profile of a 30 cm thick layer. In this scenario, the 5 cm edge areas interact with the surrounding ambient climate, while the core area exhibits increased diffusion resistance and no liquid water transport as you can see in Figure 64 below. The courses illustrate, that the multilayer case shows a fundamental different behaviour over the time which is similar to the former findings in literature (Holm, Krus und Künzle 1999). Whether this fundamental material behaviour can be confirmed by current field investigations is examined in more detail in Chapter 6.3.

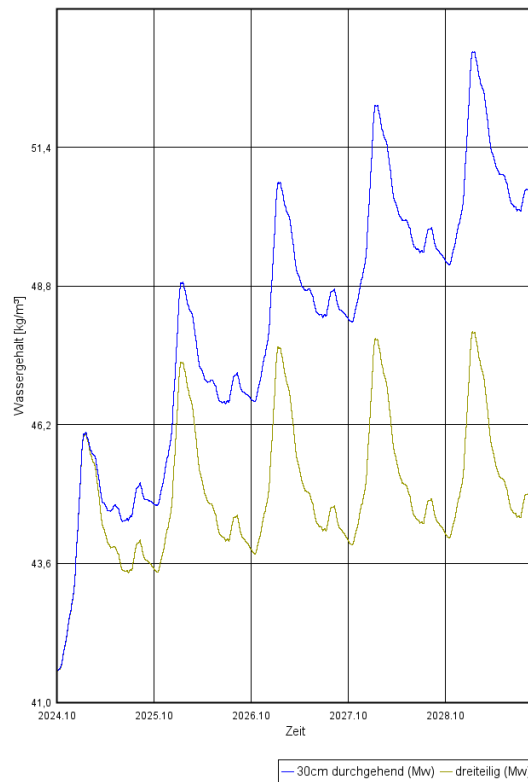


Figure 64: Simulated course of the total water content within a 30 cm thick concrete specimen over 5 years. Comparison of a homogeneous layer with liquid water transport in blue and a tripartite model in green.

6.2 Lab tests of the hygrothermal transport properties

To execute a realistic hygrothermal component simulation, reliable material properties are required for each material. The material data were obtained for the two binder types (CEM I and CEM III) as well as for the corresponding samples with different chloride contents. For each base material, 4 test specimens, prisms with a side length of 5 cm, were examined, resulting in a total of 32 specimens. Additionally, prisms for field investigations were produced, which have a length of 15 cm for a deposit in the IBP façade test facility.

The required hygrothermal material parameters are listed and shortly described in Table 32.

Table 32: Overview of the necessary hygrothermal material properties

Parameter	Unit	Measurement method
Bulk density	kg/m ³	Gravimetric measurement after drying with 40°C
Free water saturation	kg/m ³	Storage of the probes till constant mass is reached
μ- value	-	via Dry-Cup and Wet-Cup measurement
Water sorption at an equilibrium moisture content at (65 %,80 %, 93 % und 97 % RH)	kg/m ³	storage of the specimens in the corresponding ambient climate until mass constancy
A-value	kg/m ² √h	measurement of the increase in mass over time during underwater storage
True density	kg/m ³	Helium-Pycnometer

Bulk und true density are used to calculate porosity for the material data set. The A-value is used as one of the basis parameters to determine liquid transport coefficients. To create sorption isotherms (see Chapter 6.3), the equilibrium moisture contents at different RH levels as well as the free water saturation are measured. Due to the high-water vapor diffusion resistance and the resulting slow moisture uptake, the measurement of the sorption parameters takes significantly longer than for most other hygroscopic building materials. The resulting moisture contents are combined in the moisture storage function over the whole moisture range from 0 % to 100 % RH. The procedure is described in the following Chapter.

6.3 Determination of the sorption isotherms

For the hygrothermal component simulation a continuous correlation between moisture content and relative humidity within a building material is necessary (Künzel 1994).

Up to 95 % RH, the wetting process is mainly driven by sorption, while in the higher RH range also the capillary suction becomes a more and more dominant factor. Therefore, the material samples are stored at different RH levels in the climate chamber up to 95 % RH and under water for 100 % RH - in all cases until mass constancy is reached. Due to high vapor diffusion resistance and low liquid transport capacity, these measurements, which started already with delay due to the late specimen production, required long time in the project. Figure 65 shows

a schema illustrating the correlation between RH, water content and the dominant transport phenomena.

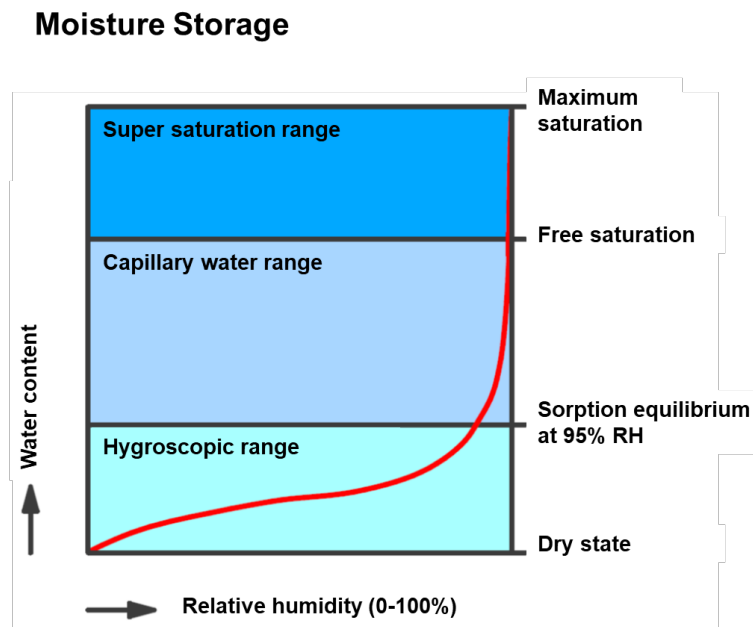


Figure 65: Exemplary moisture storage function (red curve) which describes the relation between relative humidity and water content.

The individual reference points of the sorption isotherm were determined at relative ambient humidities of 65 %, 80 %, 93 %, and 97 % RH as well as the free water saturation, which represents the water content at 100 % RH. To achieve a continuous curve from the individual reference points, an exponential function is used as fitting approach to connect the measurement points. This results in a fully defined curve for the range from 0 % to 100 % relative humidity.

Depending on whether these reference points were measured during moisture uptake (adsorption) or during drying (desorption), different curves can result. The deviation depending on individual material behaviour is the so-called hysteresis, which occurs generally in many materials, also in concretes and it is more pronounced, when they contain chlorides (Becker, Untersuchungen zum Austrocknungsverhalten von chloridhaltigem Beton und Folgen für die Korrosionsaktivität des Betonstahls 2022). Therefore, both adsorption and desorption isotherms were determined within this research project. Since the processes of local moisture increase or decrease cannot be clearly separated in practice, an average curve between the two isotherms was created and used for further hygrothermal component calculations. This result is exemplarily shown for CEM I without chloride ions in Figure 66 below.

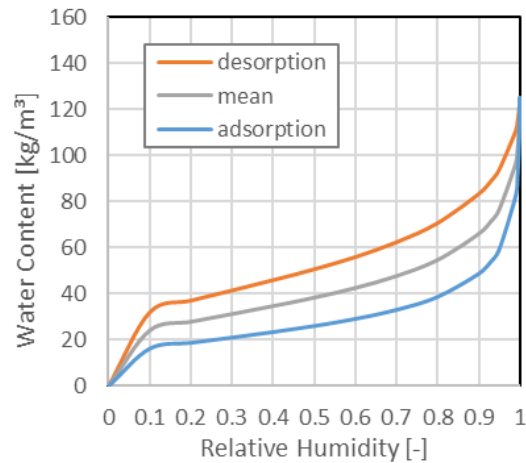


Figure 66: Exemplarily curves of the of adsorption and desorption isotherms as well as the mean curve.

Since sodium chloride (NaCl) was used for the salt-laden concrete samples, the influence of the deliquescence humidity of NaCl must also be considered. He salt leads to increased water contents above around 75 % relative humidity for NaCl (Becker 2022). This effect becomes particularly relevant at salt contents of > 2 mass percent (Becker 2022). The comparison of the sorption isotherms for CEM I with different salt contents is shown in Figure 67. However, a general increase in water content in concrete with increasing salt content could not always be clearly observed.

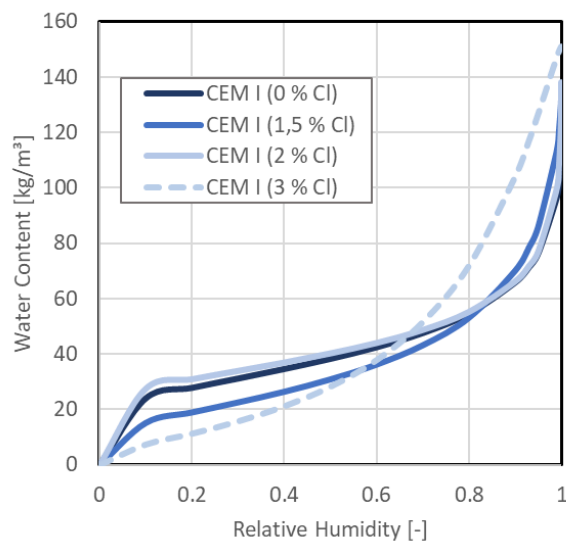


Figure 67: Comparison of the sorption isotherms for CEM I with different Cl content.

Although the water content depends on temperature and relative humidity, the temperature dependency can be neglected within the typical ambient climate range (Kast und Jokisch 1997).

Figure 68 shows an overview of the sorption isotherms for CEM I and CEM II with different chloride contents.

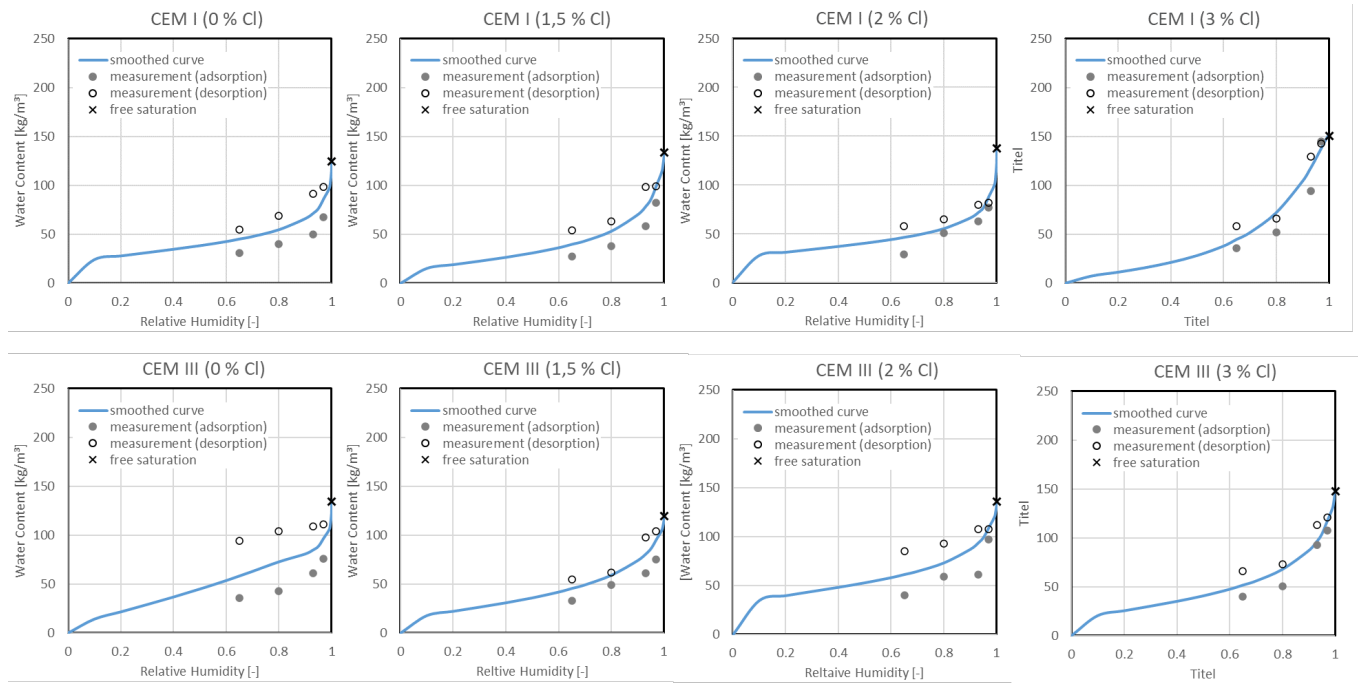


Figure 68: Overview of the sorption isotherms with and without CI content. The points are the base points for adsorption (●) and desorption (○). The blue line is the smoothed mean moisture storage function including the free saturation (x).

6.4 Determination of typical initial moisture contents

To determine the initial water content, a distinction must be made between fresh concrete and fully hardened concrete.

For fresh concrete, the mechanical strength is determined after 28 days according to for example (DIN EN 197-1 2011). However, this is a climate-dependent value; under unfavourable conditions, such as excessively high outdoor temperatures, an additional post-treatment is required in accordance with (DIN 1045 2023). The hydration of the concrete is largely but not completely finished after this period of time. This is necessary to prevent a too rapid drying of the concrete and thereby an incomplete curing.

Furthermore, for fresh concrete, the ratio of mixing water to hydraulic binder is crucial. Up to a water cement ratio of 0.4, a fully hydrated cement can be assumed, after the formation of the cement stone is complete (Backe, Hiese und Möhring 2013). A fully hardened concrete contains (in a dry ambience) hardly any additional free water and has only few pores which allow a relevant capillary transport. Higher water cement ratios initially lead to more free water in the material, and upon drying, to a larger proportion of capillary pores.

The mixing water in concrete is chemically bound over time for the hydration of the binder (cement) and is therefore no longer available as liquid water in the pores. At water cement ratio values greater than 0.4 (Backe, Hiese und Möhring 2013), liquid water remains in the initially saturated pores, which can dry out over time under appropriate climatic conditions.

For example, in the project, a Portland cement without Cl was measured with a water cement ratio of 0.55, using a mixing water amount of 193 kg/m^3 . The material data, determined from the stored samples showed a maximum saturation level of 140 kg/m^3 . This is the amount of moisture that can be introduced into the material by condensation processes or under additional pressure and thus represents the maximum possible amount. In the lab that value is determined by the material's porosity. The difference of 53 kg/m^3 between the maximum saturation and the mixing water gives the approximate amount of chemically bound water, neglecting the additional moisture introduced through diffusion during storage.

Calculating with fresh concrete is only useful if, in addition to material data, the originally used amount of mixing water and the ambient climate over the observation period are known. For normal building practice simulations and fabric concrete constructions a complete hydration can be assumed. It should be noted that the material samples were stored sufficiently long before determining the material properties. The duration should be at least 28 days, preferably several months, to achieve the highest possible degree of hydration. The assumption of an increased initial moisture content, therefore, does not consider the amount of water, necessary for the formation of the cement stone, but rather the amount of free water resulting from long-term natural weathering, depending on the location.

In hygrothermal simulations, a 10 cm concrete wall was exposed to the outdoor climate at 16 locations in Germany and Austria using the design indoor climate derived from the outdoor conditions according to (DIN 4108-3 2024). The resulting annual median water contents at the different locations are listed in Table 33 for the concretes without chlorides. These median

water contents can be used as indicator for the evaluation of renovation options at a specific location.

Table 33: Comparison of the median annual water content in dynamic equilibrium. The deviation in percent is in relation to the max occurring water content during the year at each location.

Site	Median water content at equilibrium state for CEM I		Median water content at equilibrium state for CEM III	
	kg/m ³	%	kg/m ³	%
Holzkirchen	103,8	-7	85,7	-5
Warnemünde	91,8	-7	81,8	-5
Hamburg	103,7	-6	84,0	-4
Potsdam	95,1	-7	78,8	-5
Braunlage	106,2	-7	87,0	-5
Kassel	71,3	-8	74,3	-3
Chemnitz	100,8	-7	81,5	-5
Fichtelberg	108,2	-5	90,1	-4
Mannheim	68,9	-11	73,0	-4
Stötten	104,5	-6	84,6	-5
Fürstenzell	102,3	-5	82,7	-5
Lindenberg	100,9	-5	81,9	-4
Wien	86,3	-7	75,8	-5
Graz	74,2	-11	74,3	-4
Innsbruck	64,9	-7	74,1	-3

As Table 3 illustrates, the typical water content in the equilibrium state varies significantly between 64.9 kg/m³ (Innsbruck – no rain on the western orientation) and 108.2 kg/m³ (Fichtelberg) for CEM I. For the CEM III the fluctuation is less between 73,0 kg/m³ (Mannheim) and 90 kg/m³ (Fichtelberg).

The comparison with the maximum water contents further shows the relatively narrow range of fluctuations in the equilibrium state due to the low moisture transport capacities of the concretes.

Concretes exhibit a rather inert hygroscopic material behaviour, but for example, the determination of sorption isotherms always requires the equilibrium state under the corresponding boundary conditions. The relevant standards like ISO EN ISO 12571 typically define this as achieved, when the weight change is less than 0.1 % by mass within 24 hours. Measurements within the project have shown that during drying from the initial moisture content, the longest duration to reach mass constancy as defined above was about one month. For moisture up-take through sorption in a constant climate, no more changes greater than 0.1 mass percent within one day occurred for CEM I after about one month and for CEM III after two months. This is exemplarily shown for a sample set of CEM III with a chloride content of 3 % by mass in Figure 69.

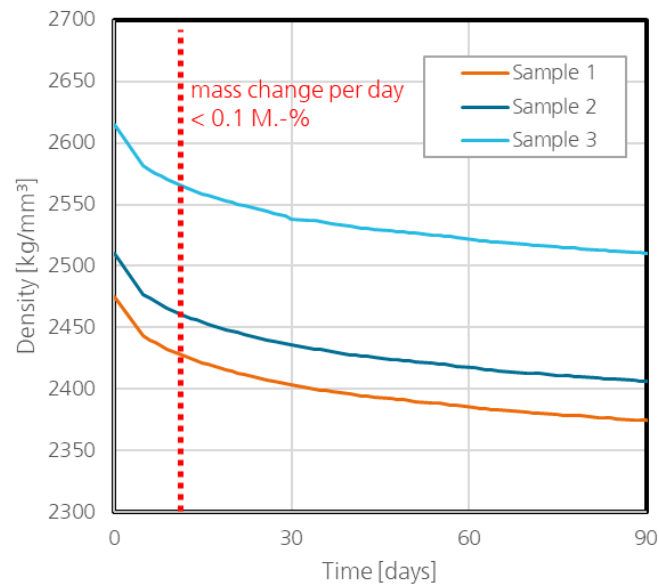


Figure 69: Development of the density of CEM III concrete containing a chloride content of 3 % by mass specimen during the drying process.

Figure 69 illustrates the change in mass during the drying process. After reaching mass constancy, defined via the standard criteria, the fluctuation is distinct smaller than the natural fluctuation between different samples of the same concrete. This is related to the small size of the samples and the thereby greater influence of the aggregate which is illustrated in



Figure 70: The varying ratio between cement matrix and aggregates of the exemplary specimens influences the material properties and thus leads to a certain variation of their parameters.

Another question was, if a stepwise drying of a specimen will lead to a different moisture level compared to a continuous direct drying. This question is also relevant for the design of new experiments and lab tests. For the investigation, three specimens were dried out from 97 % RH via one to three intermediate steps to 65 % RH. The results were compared with the result of a direct drying process. In Figure 71 the results for CEM III without CI are plotted.

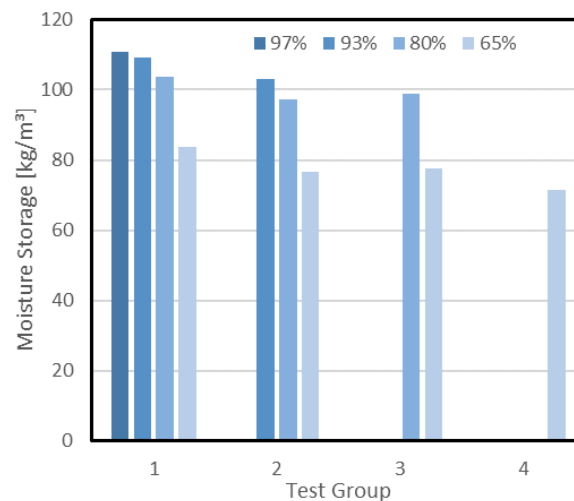


Figure 71: Moisture content of CEM III specimens under various ambient moisture levels.

When comparing the columns of different test groups in Figure 71 at the same ambient humidity, only a slight deviation is observed. This deviation is within the fluctuation range of the various samples. In Future, the number of test specimens can be reduced as the single specimens can be dried back via intermediate RH levels. Alternatively, the process can be accelerated since samples can be dried back in parallel for each sorption moisture content.

6.5 Determination of the liquid transport coefficient

The liquid transport coefficients describe the capillary transport of liquid water in a material. These coefficients cannot be measured directly; therefore, they must be determined through iterative adjustment by simulating the conducted tests to achieve a match between the observed liquid transport and the simulated transport. Since the speed of liquid transport varies significantly in the different processes of wetting (suction) and drying (distribution), the coefficients for each process are determined separately based on water absorption and drying tests.

To this end, a material file for the hygrothermal simulation program WUFI® is created using the measured properties. The liquid transport coefficients are initially set using the generated values based on the A-value. Each conducted test, water absorption test and drying test, is then separately simulated, and the simulated development of the water content is compared with the measured values. The liquid transport coefficients for each test are iteratively adjusted until the simulated values fit with the measured ones. Figure 72 illustrates this procedure schematically. The determined coefficients for suction and redistribution are shown in the final material datafile in Figure 73 to Figure 76.

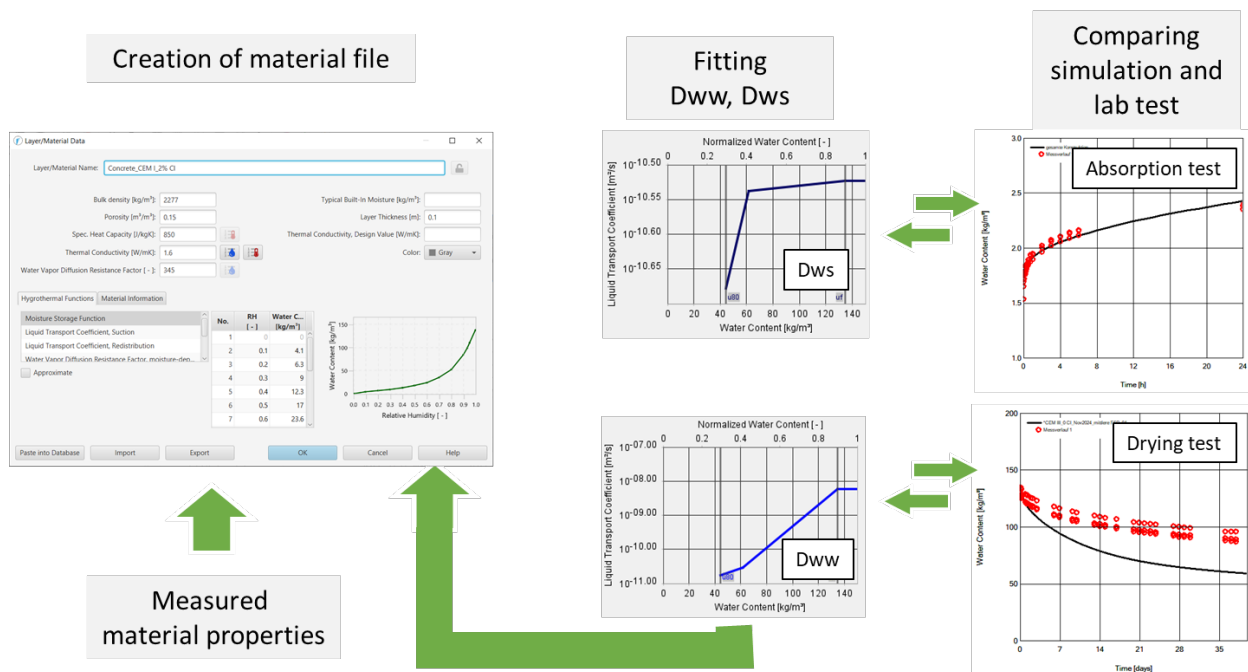


Figure 72: Determination of the liquid transport coefficients by simulation of water absorption and drying tests

6.6 Material data sets for hygrothermal simulation

6.6.1 Concrete

For eight types of concrete (two cement types CEM I and CEM II, each with four different chloride contents from 0 % and 3 %), the required hygric properties were measured in the laboratory.

For each cement type, the specimens without and with 3 % Cl-ions content were measured in detail to clarify the effect of the contained salt amount and to create material datasets for the validation (see Chapter 6.7). The porosity, water vapor diffusion resistance factor and the A-value for concrete with 1.5 % and 2 % Cl ions are calculated from the measured values for those with 0 % and 3 % Cl ions using linear interpolation based on the ion content rate. As described in Chapter 6.3, the measured values of moisture storage by the adsorption and desorption process were averaged for the use in the hygrothermal simulation, where moisture behaviour always involves a changing combination of both processes. These average values were then smoothed to optimize them for computational calculations.

These properties are summarized in Table 34. The created data sets for both types of concrete, CEM I and CEM III, with 0 % and 3 % Cl-ion contents are shown in Figure 73 to Figure 76.

Table 34: Measured material properties. The *italic* values are calculated using the measured values from the specimens with 0 % and 3 % Cl-ion content. The values for the moisture storage represent smoothed mean values obtained from measurement results by the adsorption and absorption processes.

Property		Unit	Cement type / Cl-ion contents							
			CEM I				CEM III			
			0 %	1.5 %	2 %	3 %	0 %	1.5 %	2 %	3 %
Bulk density		kg/m ³	2295	2320	2277	2264	2266	2292	2255	2287
Porosity		-	0.14	<i>0.15</i>	<i>0.15</i>	0.16	0.15	<i>0.15</i>	<i>0.15</i>	0.16
Water vapor diffusion resistance factor	3 / 50 % RH	-	395	320	294	244	617	558	538	498
	50 / 93 % RH	-	188	<i>153</i>	<i>141</i>	118	252	227	<i>218</i>	201
Moisture storage	65 % RH	kg/m ³	45.2	39.8	46.4	44.8	58.4	46.4	45.6	51.9
	80 % RH	kg/m ³	53.8	53.2	78.5	71.9	74.1	55.3	73.4	68
	93 % RH	kg/m ³	72.1	59.1	80.7	117.0	84.8	71.8	93.9	96.2
	97 % RH	kg/m ³	86.4	88.5	88.2	136.9	97.1	95.4	109.6	117.1
Free saturation		kg/m ³	125.0	133.7	138.0	145.9	135.3	120.7	136.2	148.3
A-value		kg/m ² √h	0.43	<i>0.40</i>	<i>0.39</i>	0.37	0.27	<i>0.24</i>	<i>0.23</i>	0.21

Material: *CEM I_C10 %

Property	Unit	Value
Bulk density	kg/m ³	2295
Porosity	m ³ /m ³	0.14
Water Vapor Diffusion Resistance Factor	-	395
Spec. Heat Capacity	J/kgK	850
Thermal Conductivity	W/mK	1.6
Thermal Conductivity Supplement	%/M.-%	8
Thermal Conductivity Supplement	W/mK ²	0.0002

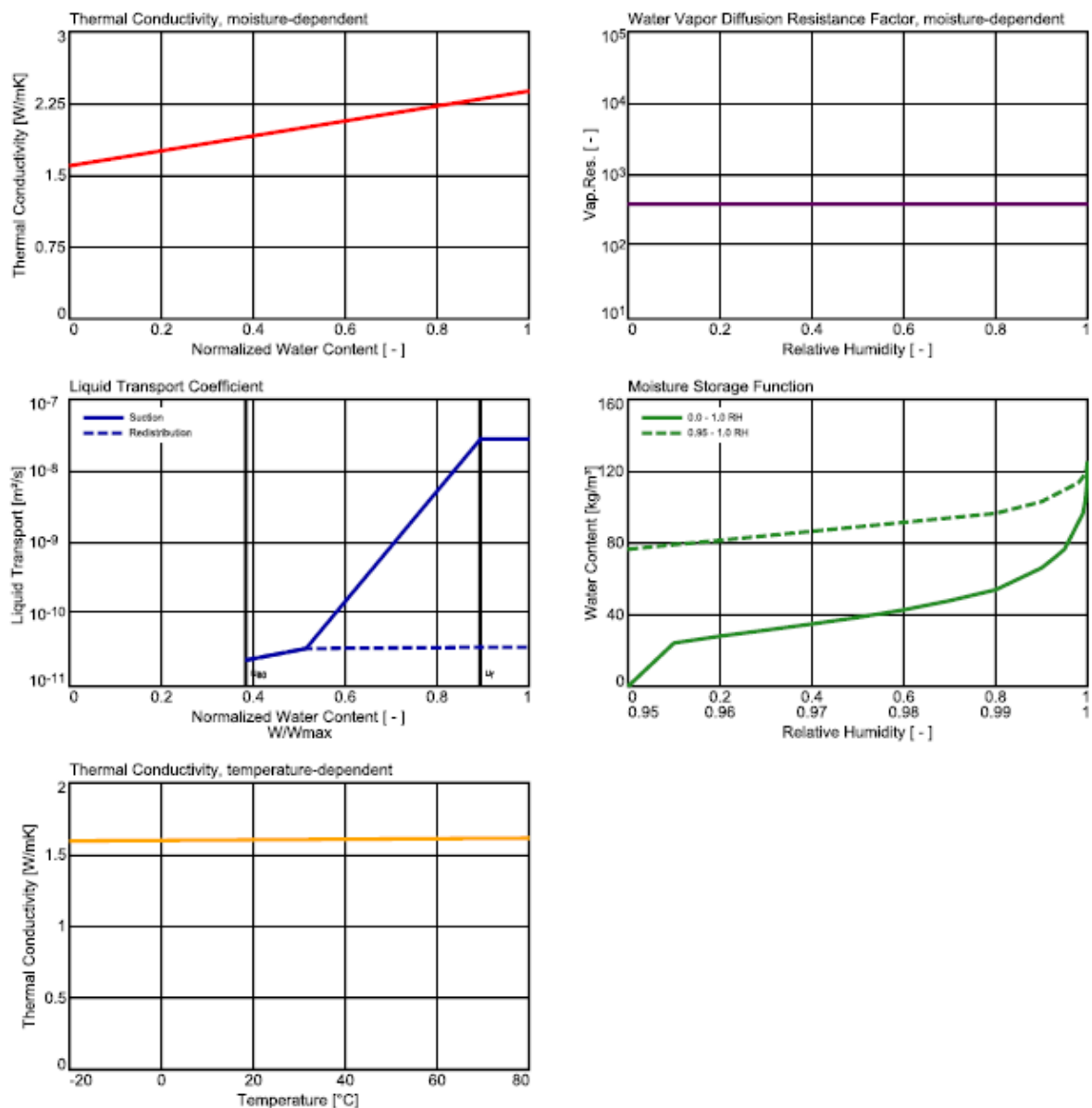


Figure 73: Material properties for concrete CEM I without chlorides (WUFI® data sheet).

Material: *CEM I_C13 %

Property	Unit	Value
Bulk density	kg/m ³	2264
Porosity	m ³ /m ³	0.16
Water Vapor Diffusion Resistance Factor	-	244
Spec. Heat Capacity	J/kgK	850
Thermal Conductivity	W/mK	1.6
Thermal Conductivity Supplement	%/M.-%	8
Thermal Conductivity Supplement	W/mK ²	0.0002

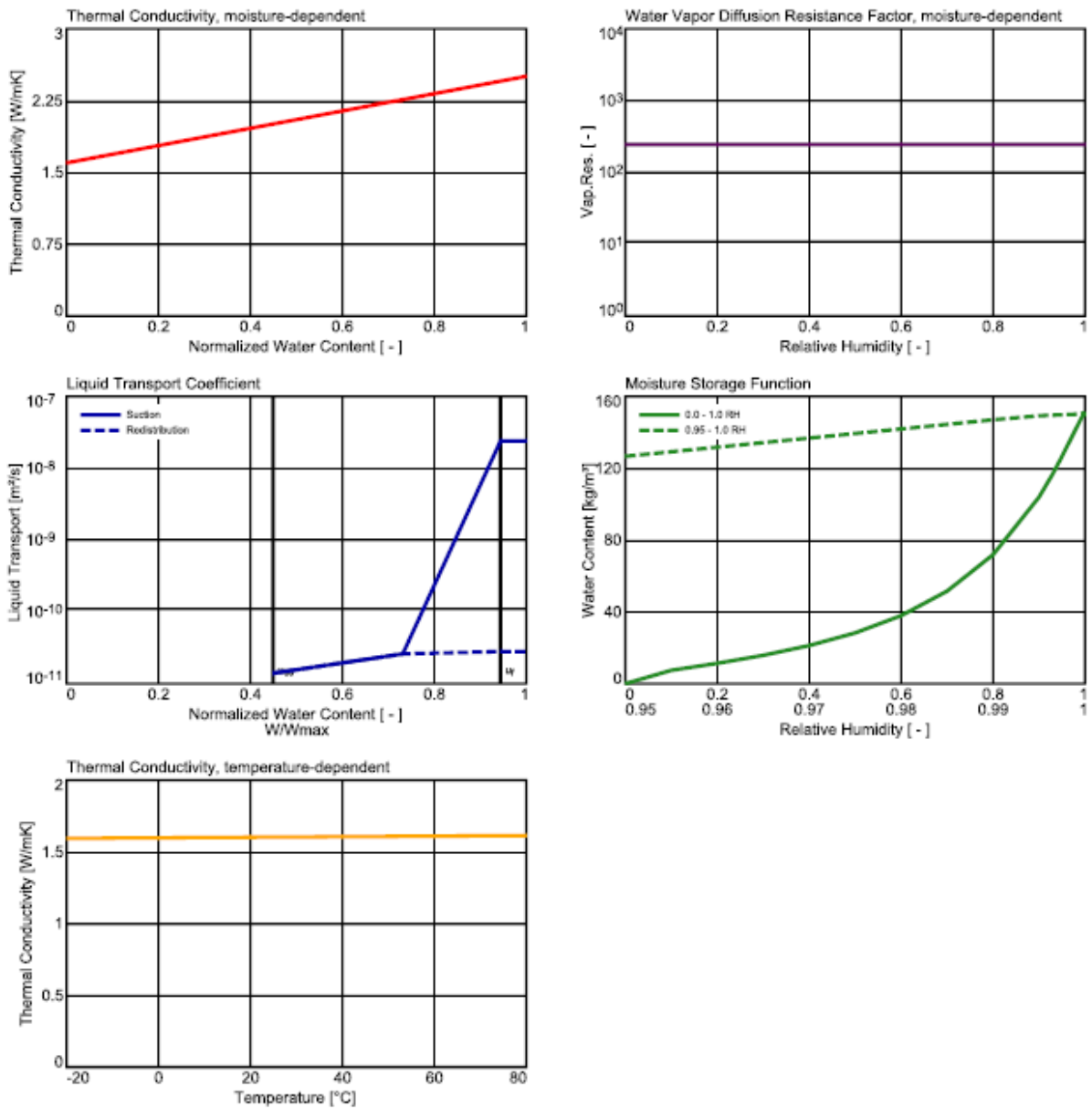


Figure 74: Material properties for concrete CEM I with 3 % Cl⁻ ions (WUFI® data sheet)

Material: *CEM III_CI 0 %

Property	Unit	Value
Bulk density	kg/m ³	2266
Porosity	m ³ /m ³	0.15
Water Vapor Diffusion Resistance Factor	-	617
Spec. Heat Capacity	J/kgK	850
Thermal Conductivity	W/mK	1.6
Thermal Conductivity Supplement	%/M.-%	8
Thermal Conductivity Supplement	W/mK ²	0.0002

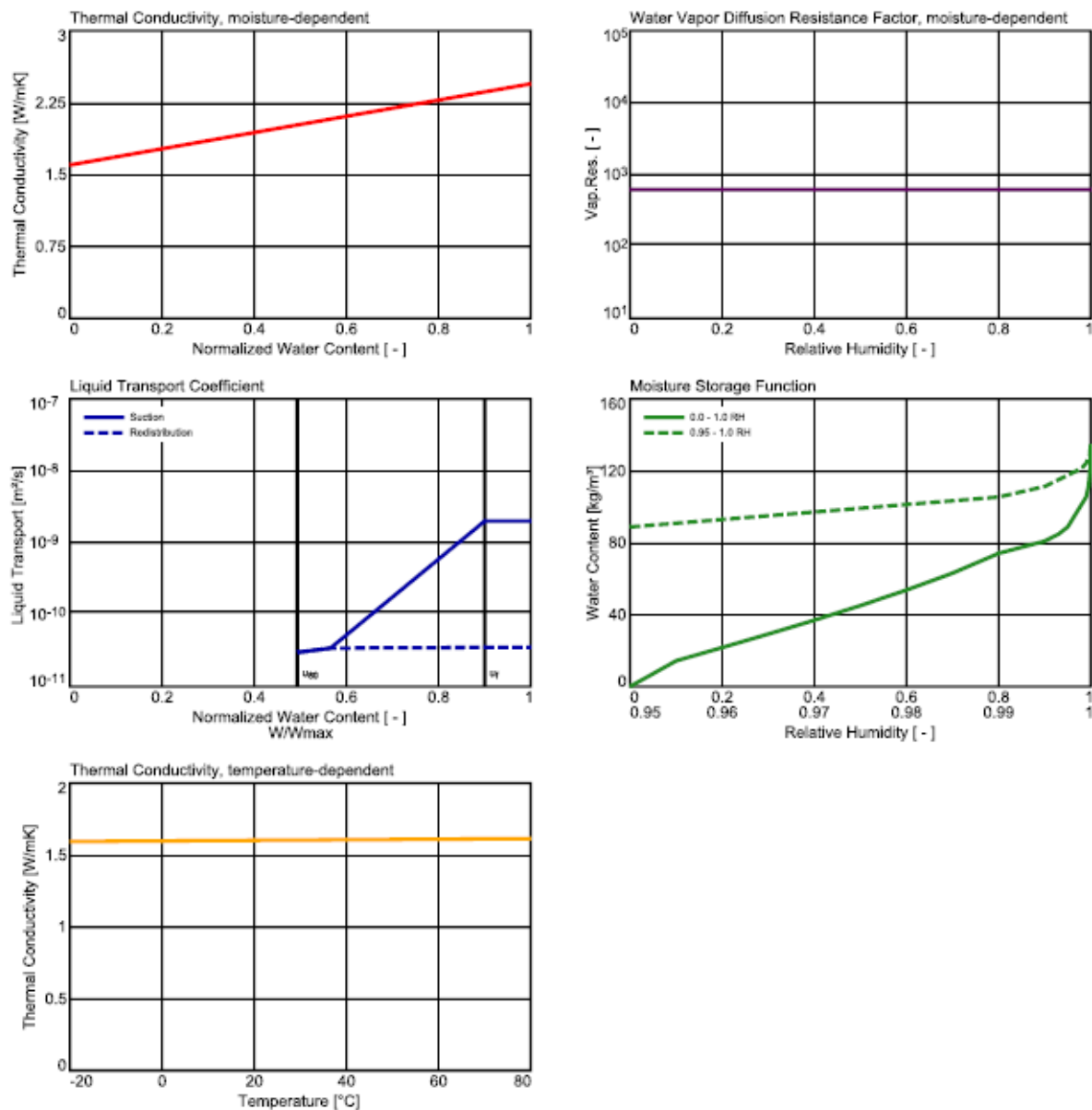


Figure 75: Material properties for concrete CEM III without chlorides (WUFI® data sheet).

Material: *CEM III_CI 3 %

Property	Unit	Value
Bulk density	kg/m ³	2287
Porosity	m ³ /m ³	0.16
Water Vapor Diffusion Resistance Factor	-	498
Spec. Heat Capacity	J/kgK	850
Thermal Conductivity	W/mK	1.6
Thermal Conductivity Supplement	%/M.-%	8
Thermal Conductivity Supplement	W/mK ²	0.0002

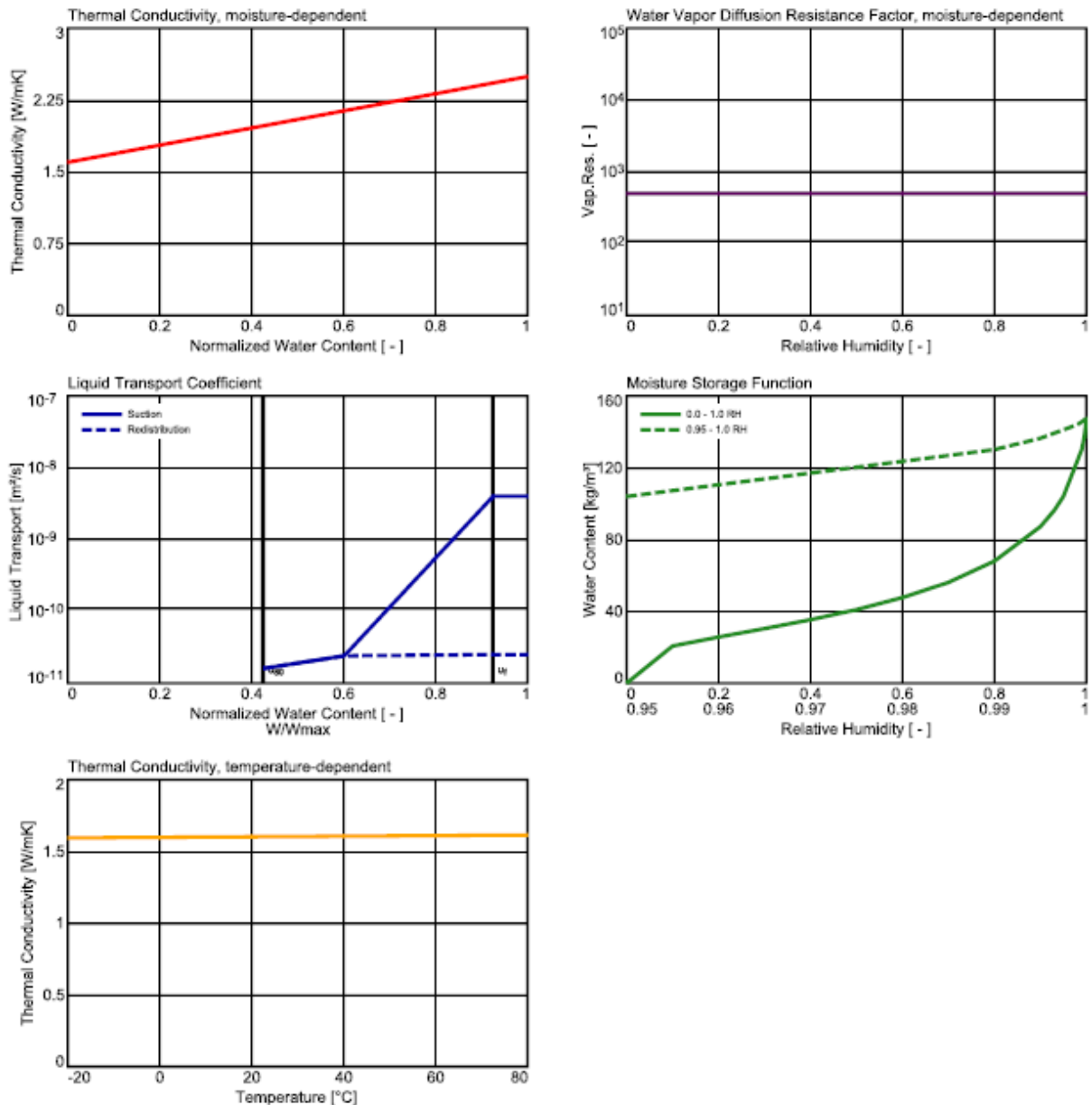


Figure 76: Material properties for concrete CEM III with 3 % Cl. ions (WUFI® data sheet).

6.6.2 Coatings

For two surface coating systems OS 5b and OS 8, the water vapor diffusion resistance factors (μ -value) were measured, and the specific s_d -values of the coating layers were calculated. The values are displayed in Table 35, the material data sheets for the use in WUFI are shown in Figure 77 and Figure 78. As coatings are usually included in WUFI® with a thickness of 1 mm, the μ -value was converted to this thickness.

Table 35: Surface protection systems - μ -values and s_d -values

Coating	μ -value [-]	Thickness [mm]	s_d -value [m]
OS 5b	3993	2.3	9 m
OS 8	170523	3.3	553 m

Material: *OS 5b

Property	Unit	Value
Bulk density	kg/m ³	170
Porosity	m ³ /m ³	0.001
Water Vapour Diffusion Resistance Factor	-	9000
Spec. Heat Capacity	J/kgK	2300
Thermal Conductivity	W/mK	2.3
Thermal Conductivity Supplement	%/M.-%	-
Thermal Conductivity Supplement	W/mK ²	0.0002

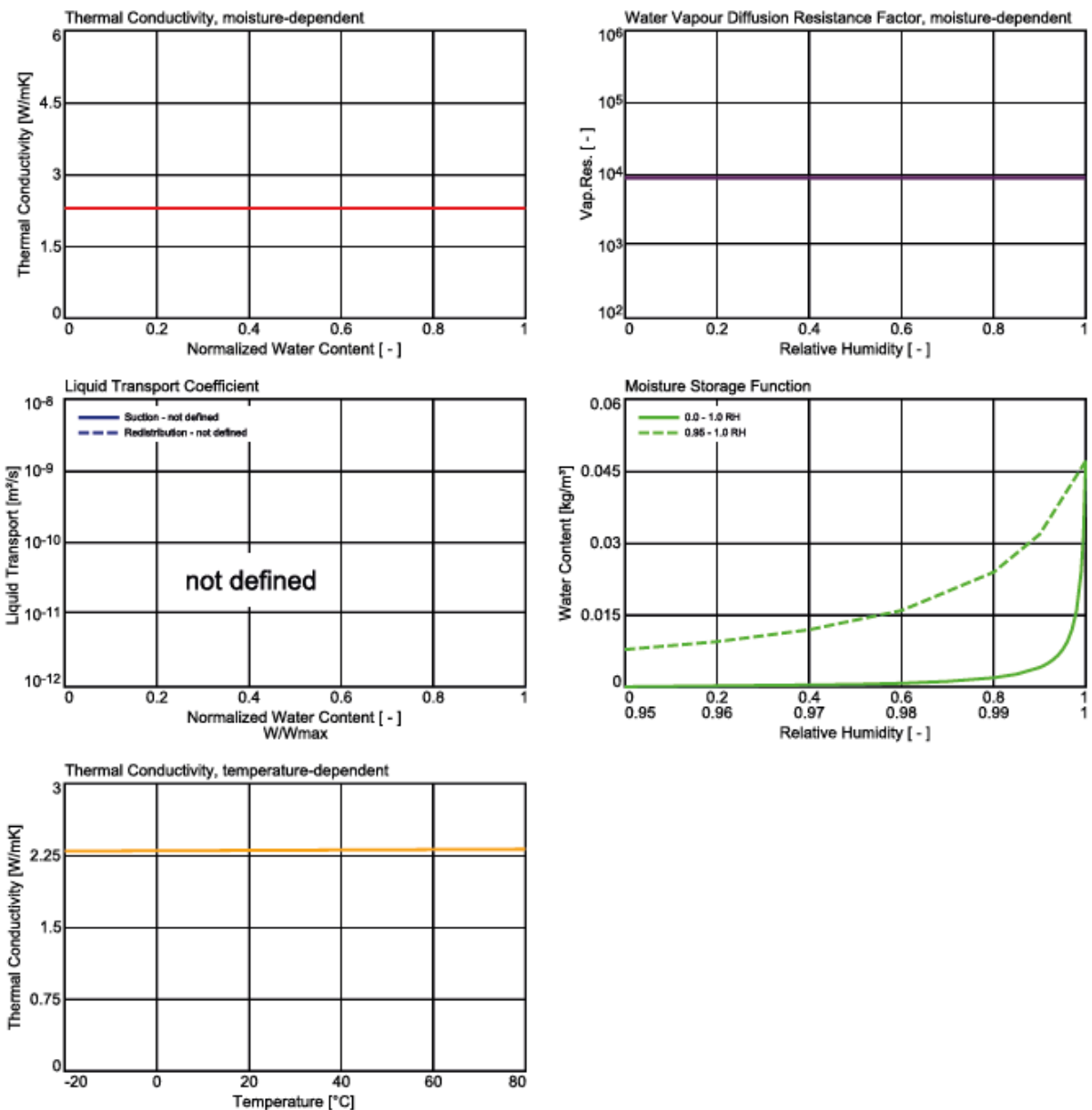


Figure 77: Material properties of the surface coating OS 5b (WUFI® data sheet).

Material: *OS 8

Property	Unit	Value
Bulk density	kg/m ³	666
Porosity	m ³ /m ³	0.001
Water Vapour Diffusion Resistance Factor	-	553000
Spec. Heat Capacity	J/kgK	2300
Thermal Conductivity	W/mK	2.3
Thermal Conductivity Supplement	%/M.-%	-
Thermal Conductivity Supplement	W/mK ²	0.0002

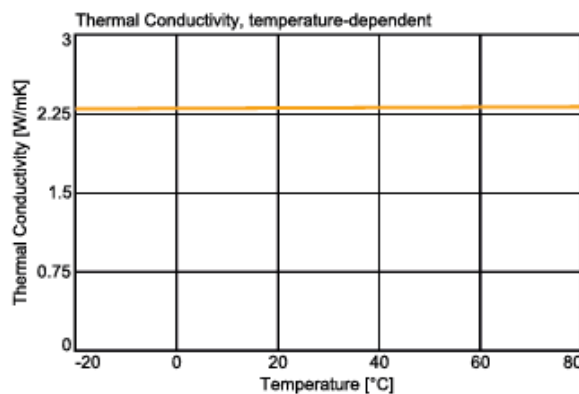
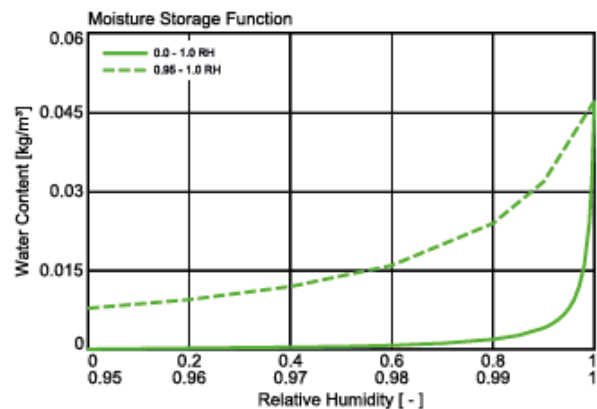
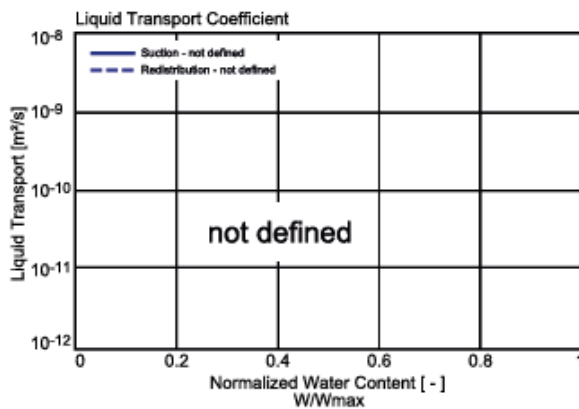
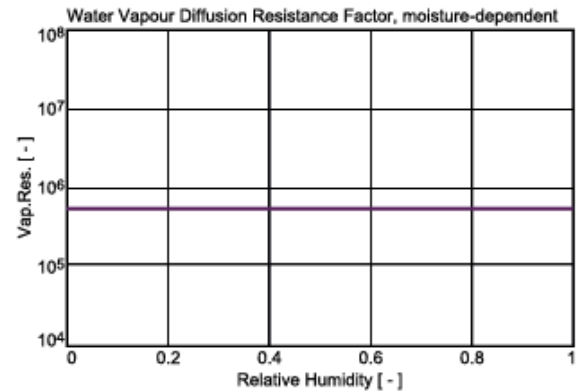
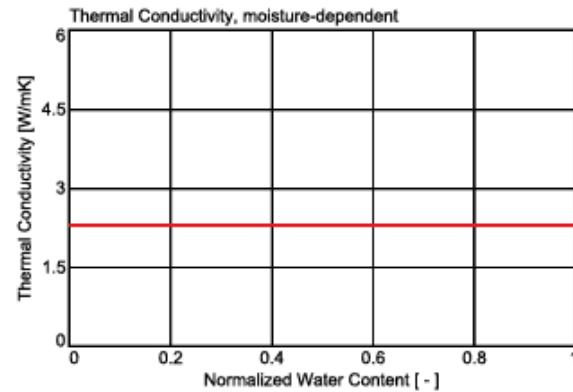


Figure 78: Material properties of the surface coating system OS 8 (WUFI® data sheet).

6.7 Validation of the new concrete material model

In this Chapter, it will be examined whether the tripartite material approach described in Chapter 5.1 proves is effective for hygrothermal component simulation in practical applications. This approach represents the formation of a capillary barrier in the core area of the component and thus account for the effects described by (Holm, Krus und Künzels 1999).

For this purpose, field measurements of concrete prisms will be compared with the corresponding simulation results. The concrete prisms were installed on the outdoor test site in Holzkirchen in a vertically oriented west-facing wall at a height of approximately 2 meters above ground level (Figure 79).



Figure 79: Field test facility in Holzkirchen with west orientation with strong driving rain load Left: CRUFI specimens are exposed at 2m height. Right: detailed built in situation.

The prisms are in contact with the outdoor climate with the exposed surface and the indoor climate at the back side, while the five lateral surfaces are sealed and insulated what leads to a largely one dimensional exposure situation. Prior to the experiment, the samples were conditioned in a climate chamber at 23 °C and 50 % relative humidity.

For the simulation, the specifically measured material datasets of that concrete were used. As outdoor climate the measured data for the year 2024 including the driving rain load measured directly at this test façade are used. The water content of the specimens was measured non-destructively using an NMR scanner. For each combination of binder and CI content, three samples were under observation. For CEM III without chlorides, the resulting water content profiles of the specimens are shown in Figure 80 for an exposure period of 3.5 months and in Figure 81 for an exposure period of approximately 5 months.

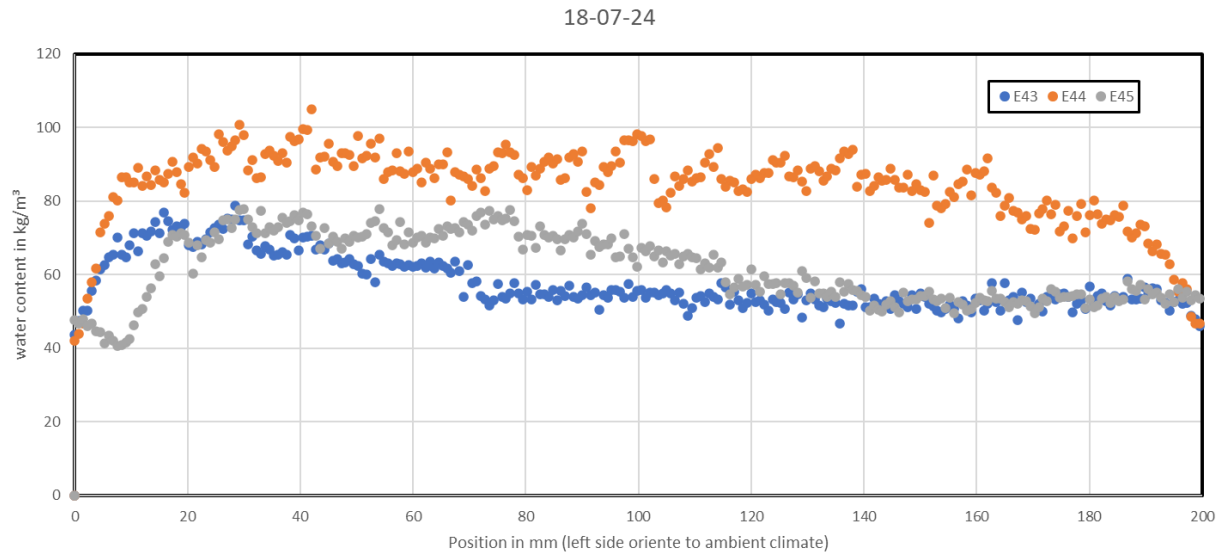


Figure 80: Measured water content profiles after 3.5 months in the three test prisms with CEM III binder without chlorides; outdoor climate on the left.

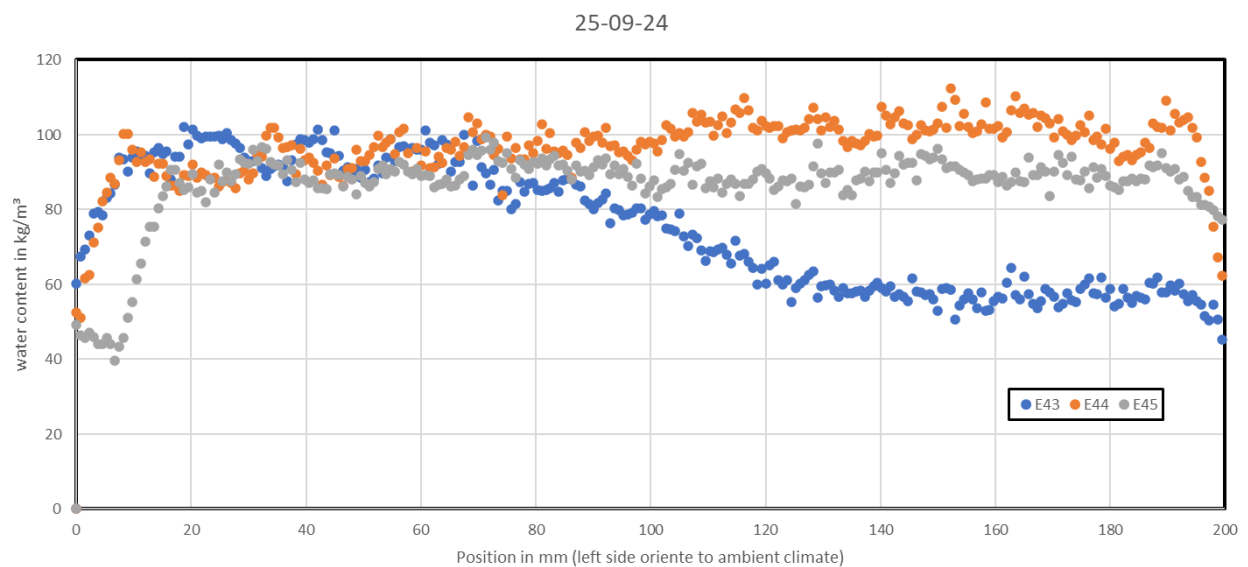


Figure 81: Measured water content profiles after 5 months in the three test prisms with CEM III binder without chlorides; outdoor climate on the left.

As shown in Figure 80 and Figure 81, the three specimens show a pretty different behaviour. While E43 shows a plausible water content profile as expected, with higher water contents near the exterior and a progressive increase in moisture throughout the entire cross-section over the time, the other two samples show a rather uniform increase in moisture across the cross-section. This seems to be not plausible and may be caused by a faulty lateral sealing of the specimen, resulting in an additional moisture uptake through the side surfaces. Therefore, only specimen E43 is used for comparison with the simulation data. This comparison is presented below in Figure 82.

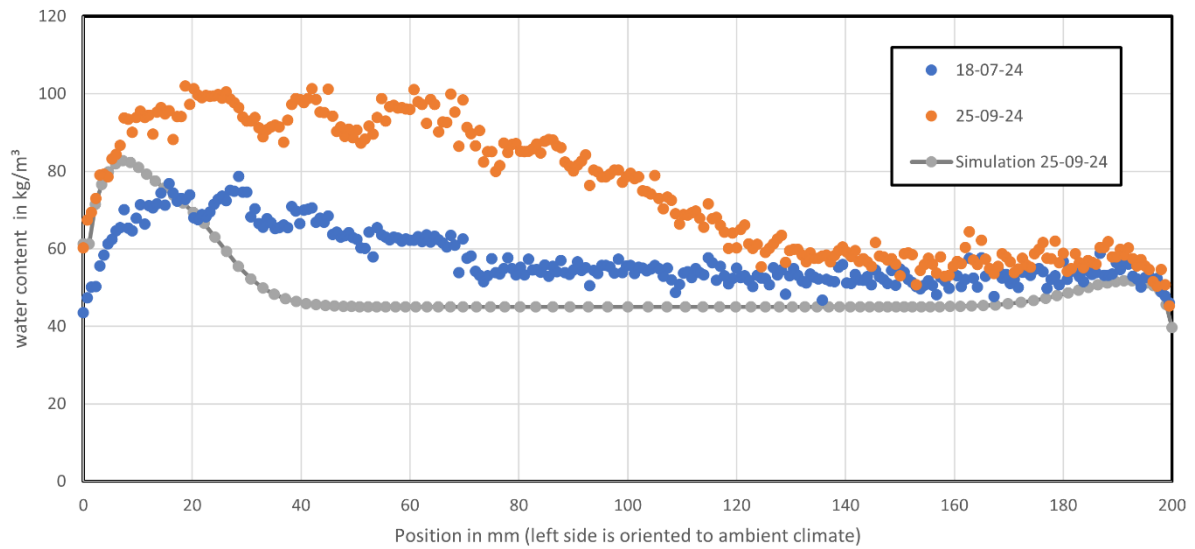


Figure 82: Comparison of the simulated water content profile end of September (grey) with the measured water content profile of CEM III binder without chlorides in July and September.

Figure 82 shows that the simulated water content at the edges of the specimen increases slower compared to the measurements. After approximately 5 months, the measured prism is saturated to a depth of approx. 8 cm, whereas in the simulation this is only the case up to a depths of 4 cm. Also the measured water contents are with 100 kg/m³ higher than the simulated ones with only about 85 kg/m³.

The next cases under observation are the specimen with CEM III binder and 3 % Cl. The investigation starts again with a comparison of the water content over the cross-section of the three specimen after 3.5 months and 5 months in Figure 83.

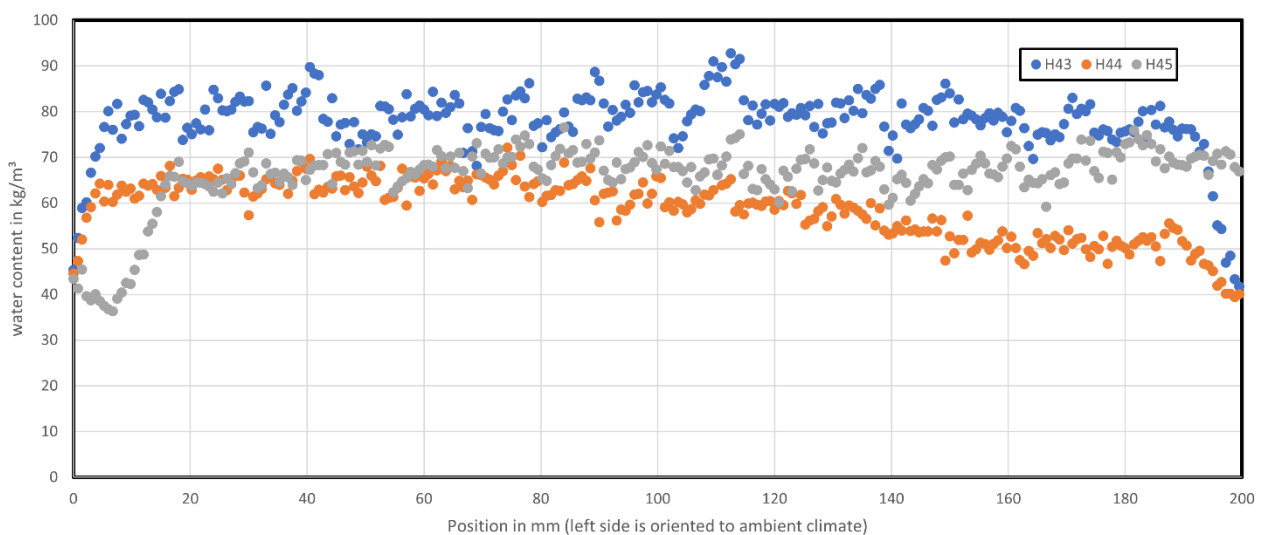


Figure 83: Measured water content profiles after 3.5 months in the three test prisms with CEM III binder with 3 % chlorides; outdoor climate on the left.

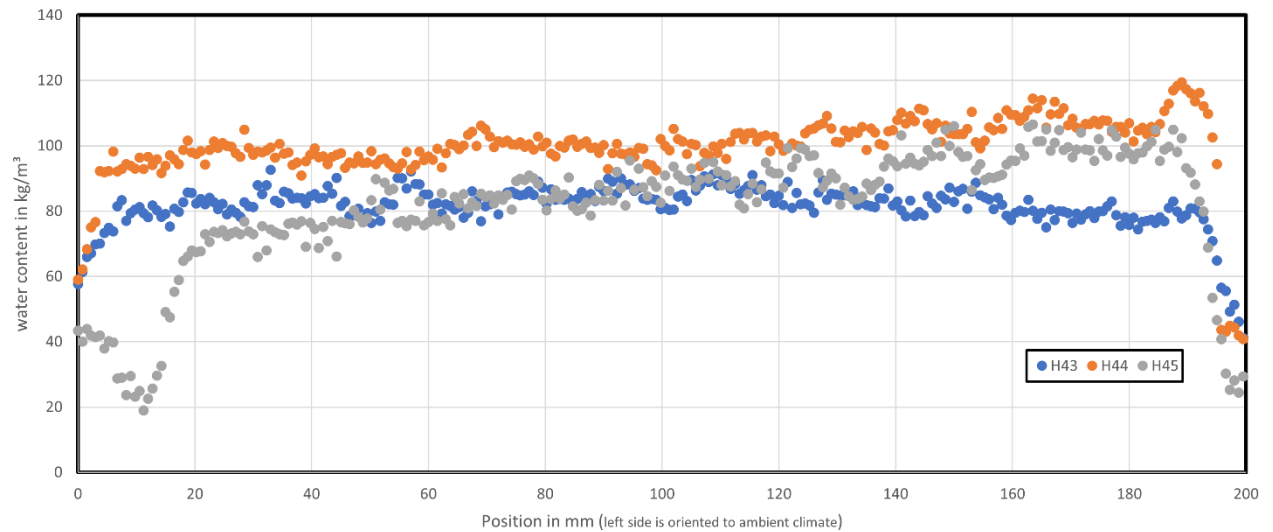


Figure 84: Measured water content profiles after 5 months in the three test prisms with CEM III binder with 3 % chlorides; outdoor climate on the left.

Unfortunately, the lateral sealing of the test specimens was partially damaged versus the end of the test period. This can be seen in Figure 85. The resulting water content suggest a lateral moisture penetration and humidification of the test prisms. Especially the uniform moisture distribution of the samples E 43 and E 44 can hardly occur in case of a tight sealing.

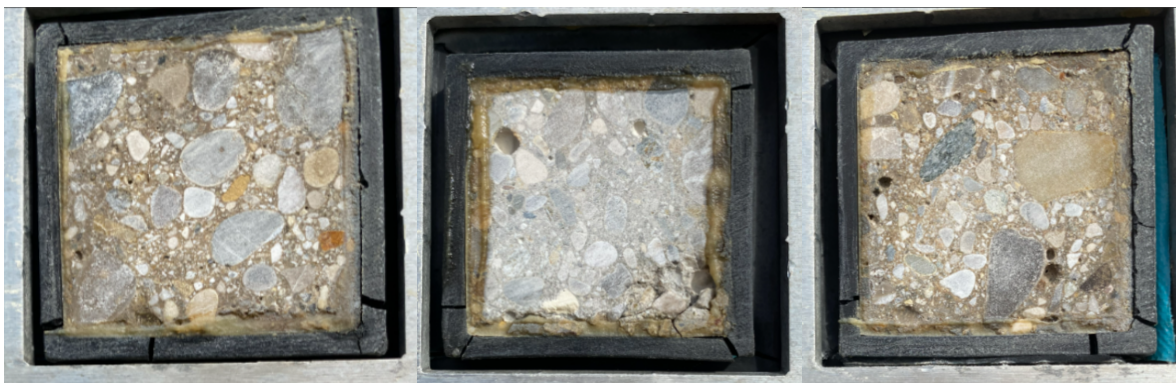


Figure 85: Exemplary cracks in the outer sealing layer of the test specimens.

Therefore, in that case no comparison between simulation and measurement is possible. point. For orientation, the simulated moisture profiles after five months are presented in Figure 86 and show as expected the climate influence on both sides which is dominant at the surfaces but not in the middle of the specimen.

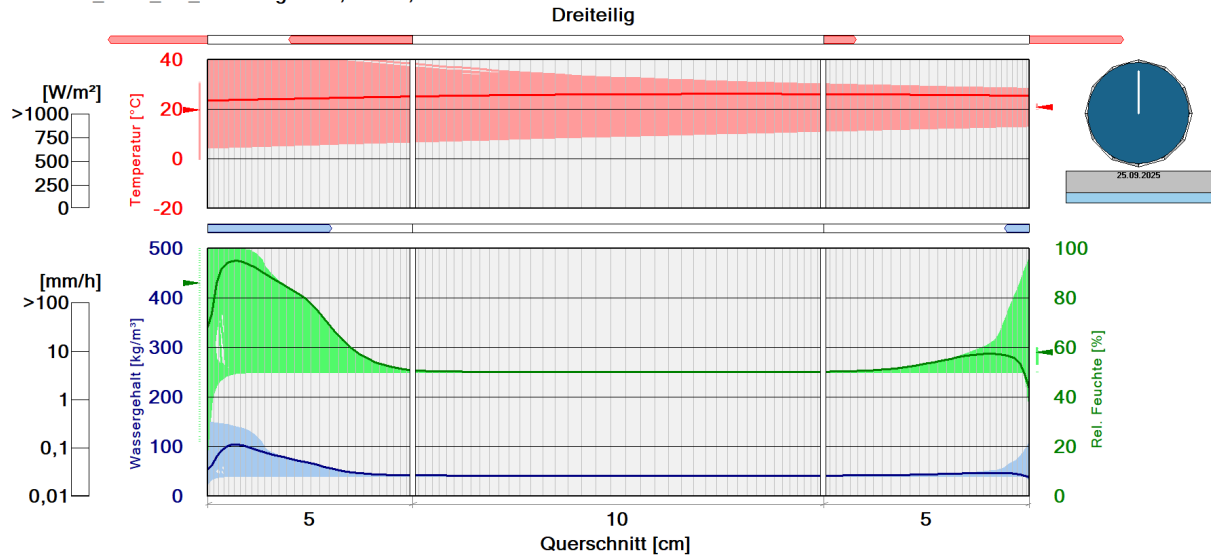


Figure 86: Visualization of the hygrothermal component results as of September 25, 2024. The water content profile is shown in blue, the relative humidity profile in green, and the temperature profile in red. The left side is exposed to the outdoor climate, while the right side is exposed to the indoor climate.

As Figure 86 illustrates, the measured profiles in Figure 84 can hardly result from a one-dimensional moisture entry only via the surfaces of the specimen. Nevertheless, to assess the change in component behaviour with chlorides in the concrete, the measurements for CEM I with a 3 % Cl_i content is additionally compared with the simulation results in Figure 87.

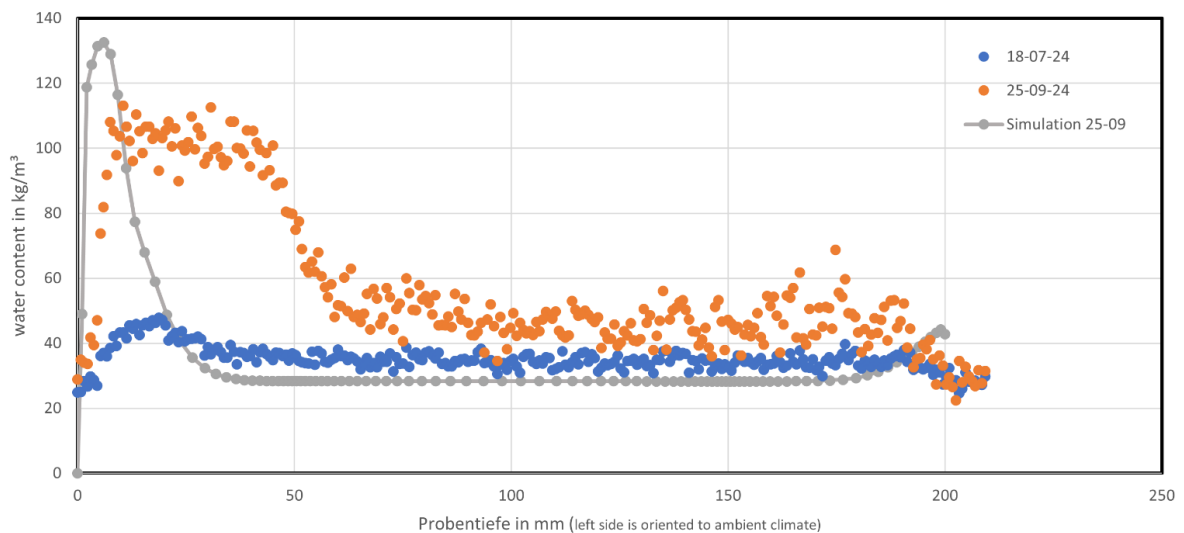


Figure 87: Comparison of the measured water content profiles in the CEM I sample prisms with 3% Cl_i in July (blue) and September (orange) with the profiles from the hygrothermal component simulation in September (grey).

As illustrated in Figure 87, the modelling approach shows the desired lower water content in the core region for the increased Cl content, while the edge areas indicate a significant in-

crease in water content. However, the simulated water content in the core region is on average about 20 kg/m³ lower than the measured value at the same time. At the edge facing the ambient climate, the simulation shows a penetration depth about 3 cm less, but also a higher maximum water content.

In summary, it can be stated that the three-parted structure of the concrete can reproduce the long-term material behaviour under natural weathering, including the formation of a capillary barrier in the center of the concrete. The penetration rate with respect to redistribution in the outer area appears to be too slow. Further adjustment of the liquid transport function for the redistribution mechanism may be necessary here. When comparing measurement and simulation, the simulated penetration depth is about half that observed in the measurements.

In addition, numerous NMR measurements showed an unrealistically uniform distribution. As a result, the measured moisture level appears relatively uniform across the entire cross-section, whereas in the simulation, the extreme values occur at the outer edges. This is presumably due to a lateral sealing that was not permanently effective. Before conducting new measurements, the samples should be conditioned to a uniform water content and a permanent sealing of the lateral areas should be ensured. This could be achieved, for example, by applying a hydrophobic wax coating. Furthermore, a measurement period longer than six months would be advisable for validating the new simulation model.

7 Development of a corrosion prediction model and corrosion maps for different concrete types

For the evaluation of hygrothermal simulation results, Fraunhofer IBP has developed a post-process module to predict the corrosion progress in building materials depending on exposure conditions and embedding materials. This model WUFI Corr is available for free and currently applicable for embedding materials from heritage buildings like historic mortars and masonry. Up to now, the prediction of corrosion in concrete is focused on carbonized normal concrete. In the current project, based on corrosion measurements, the model shall be extended to predict also the corrosion progress of metal in different concretes containing chlorides.

At the time of reporting, the data from the corrosion measurements were not yet fully completed. Therefore, the evaluation and the modelling were performed based on the available preliminary data. In particular, the moisture profiles in the specimens, associated with the corrosion measurements, were not yet available. As a result, the correlations calculated below could not yet be finally validated. The results obtained appear somewhat optimistic and will therefore be further validated once all the necessary measurement data is available.

7.1 Development and Validation of a transient model for the prediction of corrosion in concretes

Measurements of the corrosion current density on concrete specimens with different chloride contents (see Chapter 4 and 8.1) were used to generate corrosion maps for the prediction of corrosion in the specific embedding salt loaded concretes. For the specimen storage conditions, weight as well as corrosion current were measured. After an initial conditioning at 20 °C and 98 % RH over one year, the specimens were stored alternating under two climatic conditions (20 °C and 40 °C both at 55 % RH) for about one month at a time to measure the corrosion current during the drying process of the concrete. After the second warm period, the corrosion had largely subsided, and the specimen were subsequently kept at 20 °C and 55 % RH. The experimental setup is described in detail in Section 8.1. Figure 88 shows representative measured data for CEM I and CEM III, both with a chloride content of 3 % Cl. A diagram including all measured specimens is shown in Section 8.1 Figure 113.

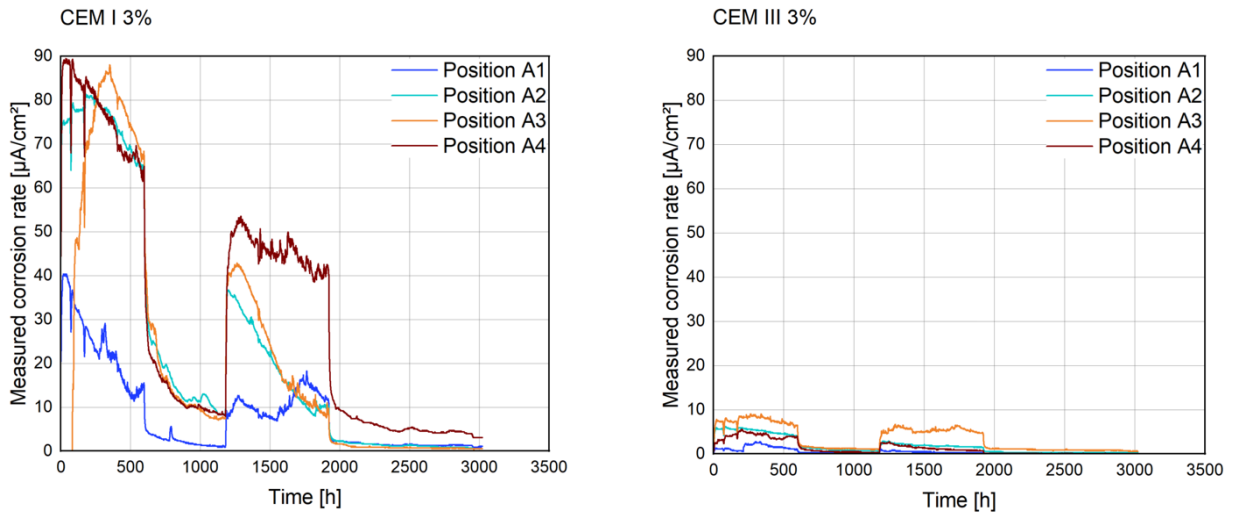


Figure 88: Corrosion current over time for the CEM I (left) and CEM III (right) specimens with a chloride content of 3 % measured at 20 °C and 40 °C both at 55 % RH.

To determine the relative humidity in the different areas, the exposure of the test specimens was simulated using WUFI 2D in order to analyse the drying behaviour. However, due to the lack of reliable measurement data, the resulting water contents and corresponding relative humidities could not yet be conclusively verified or validated.

The material parameters specified in Chapter 6 were used for this purpose. A climate file showing the temperature and relative humidity was created for the climatic conditions used in the laboratory. The positions for the evaluation (see Figure 90) were selected according to the location of the anodes (see Figure 89) in the test specimens.

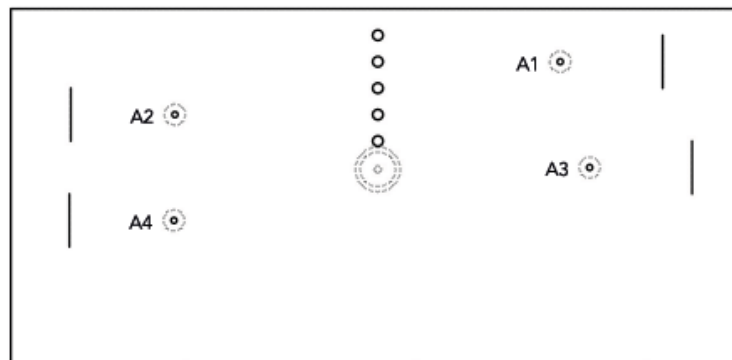


Figure 89: Setup of the test specimen, positions of anodes (A1 – A4).



Figure 90: Positions for the evaluation of the relative humidity corresponding to the anodes (see Figure 89).

The diagrams in Figure 91 show the relative humidity curves determined for CEM I and CEM III with a chloride content of 3 % for the position of anode A2.

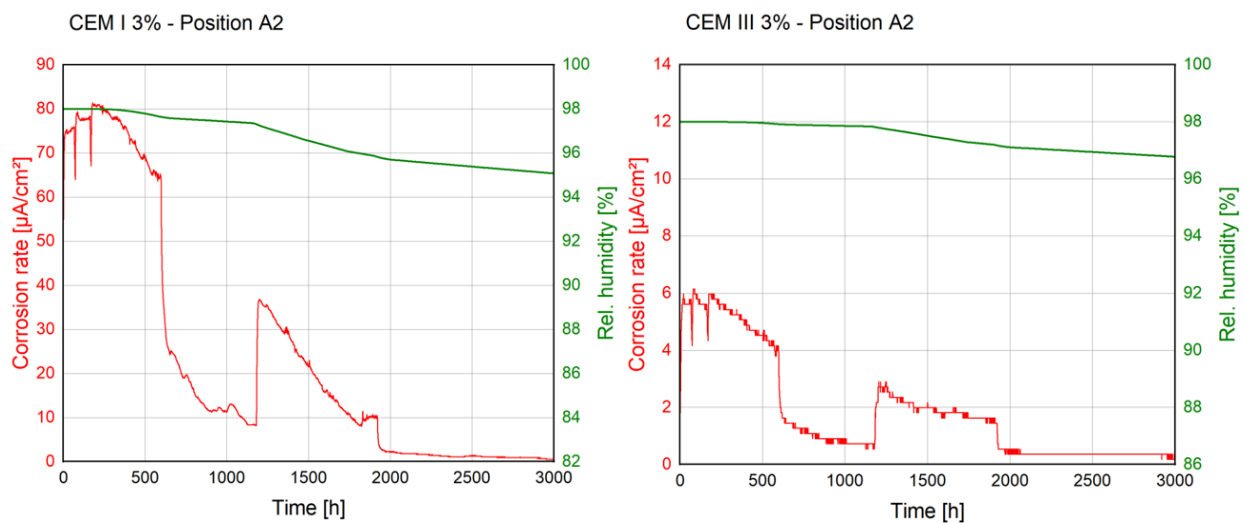


Figure 91: Corrosion current and corresponding RH calculated with WUFI 2D over time for the CEM I (left) and CEM III (right) specimen with a chloride content of 3 %

Figure 92 shows the corrosion current plotted against relative humidity. A considerable scatter of the results can be observed. In addition, the data suggest that corrosion virtually ceases at relative humidity values above 95 % RH. This threshold appears unexpectedly high, as practical experience indicates that chloride-induced corrosion may already initiate at relative humidities of around 70 % RH. Consequently, this approach is likely to yield overly optimistic results. As discussed above, the water content of the test specimens was not yet available at the time of evaluation, and the calculated relative humidity values could therefore not be validated. As a result, the corrosion map shown below is only considered valid for relative humidities above 95 % RH.

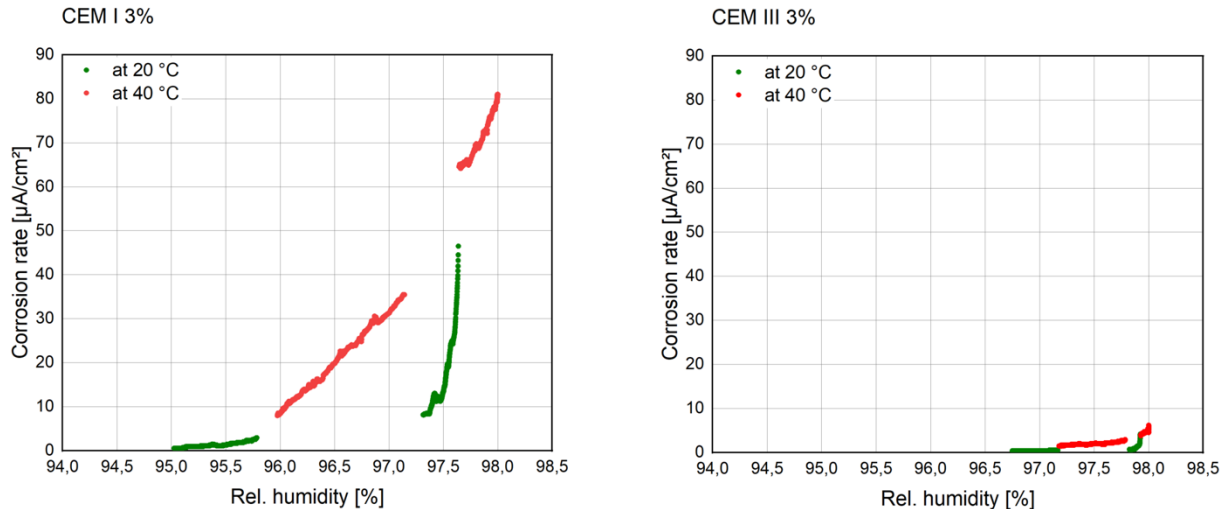


Figure 92: Corrosion current over relative humidity for CEM I (left) and CEM III (right) specimen with a chloride content of 3 %

According to Arrhenius' equation, the relationship between corrosion rate and relative humidity can also be determined for other temperatures. The corresponding procedure is described in Section 8.1.3.

The corrosion maps for the respective embedding materials were derived in accordance with (Marra, Zirkelbach und Künzle 2015) based on the measured corrosion currents and the temperature and relative humidity profiles calculated using WUFI 2D. For this purpose, the following formulation was applied:

$$i_{corr,1} = d_1 \vartheta \cdot e^{(a_1 \cdot \vartheta + b_1 \cdot RH + c_1)}$$

where	i_{corr}	corrosion rate at ϑ , RH [A]
	ϑ	temperature at reinforcement level
	RH	relative humidity at reinforcement level
	a_1, b_1, c_1 and d_1	regression parameters

This formulation has already proven suitable in previous assessments of corrosion in concrete. Figure 93 shows, as an example, the corrosion map for CEM I with a chloride content of 3 %. Further verification and validation of the model are required, particularly with regard to correlating the predicted corrosion progress with detailed measurements of moisture content in the different regions of the concrete. In particular, the low corrosion rate predicted below 95 % RH appears somewhat optimistic for chloride-induced corrosion. However, that data obtained within the scope of this project are not sufficient to draw a definitive conclusion on this issue.

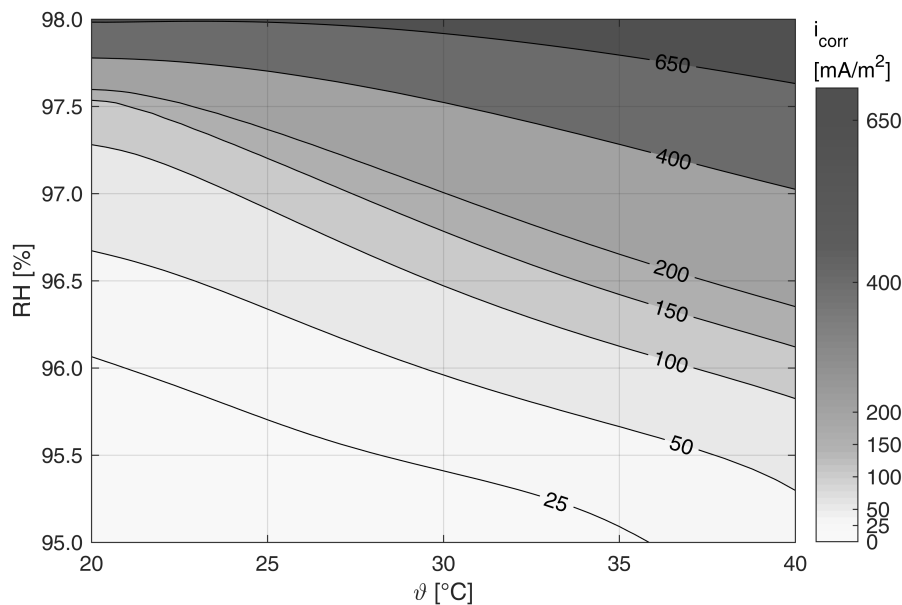


Figure 93: Corrosion map for CEM I 3 % Cl.

It may also be worthwhile to investigate whether deriving the corrosion maps using an Arrhenius-based approach would provide improved results. However, since sufficient data could not be obtained within the duration of the project, a final assessment of the suitability of this approach was not possible. Further investigations are therefore recommended to determine whether an Arrhenius-based formulation provides a more reliable description of the corrosion behaviour and improves the predictive accuracy of the model.

7.2 Effect of coatings, paints or insulation on moisture level at different exposures

The appropriate use of coatings and paints can help regulate water absorption and drying in building components. The proper application of insulation layers influences the thermal behaviour of building components and, consequently, their drying and moisture uptake behaviour. The aim is to remove moisture from the component as quickly as possible so that corrosion activity ceases.

Coatings and paint

The *Deutscher Ausschuss für Stahlbeton DAfStb* (German Committee for Reinforced Concrete) and the *Deutsches Institut für Bautechnik DIBt* (German Institute for Building Technology) list various methods for the repair of concrete components in their regulations. One of the methods used in cases of chloride-induced reinforcement corrosion, in which the damaged existing concrete does not need to be removed but is instead coated, is the W-CI (DAfStb 2001) or 8.3 (DIBt 2020) method. The state-of-the-art building regulations classification and current research on this method are discussed by (Mayer, et al. 2020). This repair method carries a significantly higher risk of failure than conventional concrete repair methods,

as the chlorides remain within the structure. Therefore, effectiveness must be proven by creating appropriate test procedures and measuring corrosion currents.

Studies investigating the effectiveness of this method are being conducted by (Wunderle-Beck, et al. 2022). They mention that removing the moisture content of concrete by applying a coating can be a very slow process. Particularly in the case of dense concrete and thick concrete coverings, it must be assumed that it will take a correspondingly long time to reduce the moisture content to the level of the reinforcement, during which time the corrosion process will continue largely unhindered. Questions regarding the prospects of success of such repair measures could, in future, be clarified in advance by means of hygrothermal simulations using the extended material models and corrosion parameters obtained within the framework of this project.

To illustrate the influence of two coating systems frequently used for the sealing and retrofitting of concrete on drying behavior, two-dimensional hygrothermal calculations were performed with WUFI 2D on a reinforced concrete column. The columns are assumed to be covered by a roof but open at the sides, so it is exposed only to outdoor temperature and relative humidity. For the location, the hygrothermal reference year of Holzkirchen from the WUFI database is used.

The column is assumed to have dimensions of 40 cm by 40 cm. The material properties of chloride-free CEM I 42.5 N concrete determined in Chapter 6 were used for the concrete. As the initial moisture condition, the equilibrium moisture content at 90 % RH (corresponds to a water content of 69 kg/m³) was assumed. As coatings, the surface protection systems OS 5b (s_d -value 9 m) and OS 8 (s_d -value 533 m) are used (see also Chapter 6.6.2).

The distribution of the water content in both columns after an exposure of 10 years is shown in Figure 94. While the column coated with OS 5b shows significant drying with water contents at the edges of about 50 kg/m³, with the OS 8 coating, the water content hardly changes. Here the edges are at about 67 kg/m³.

Figure 95 shows the areas for which the courses of water content are evaluated. The corresponding courses are displayed in Figure 96. The column coated with OS 5b, which is much more permeable to water vapor diffusion than OS 8, dries over the simulated period, although no significant temperature gradient is expected in the free-standing column and therefore only low vapor pressure gradients develop. This shows that depending on the vapor diffusion resistance of the surface protection system the column may either dry out or retain trapped moisture.

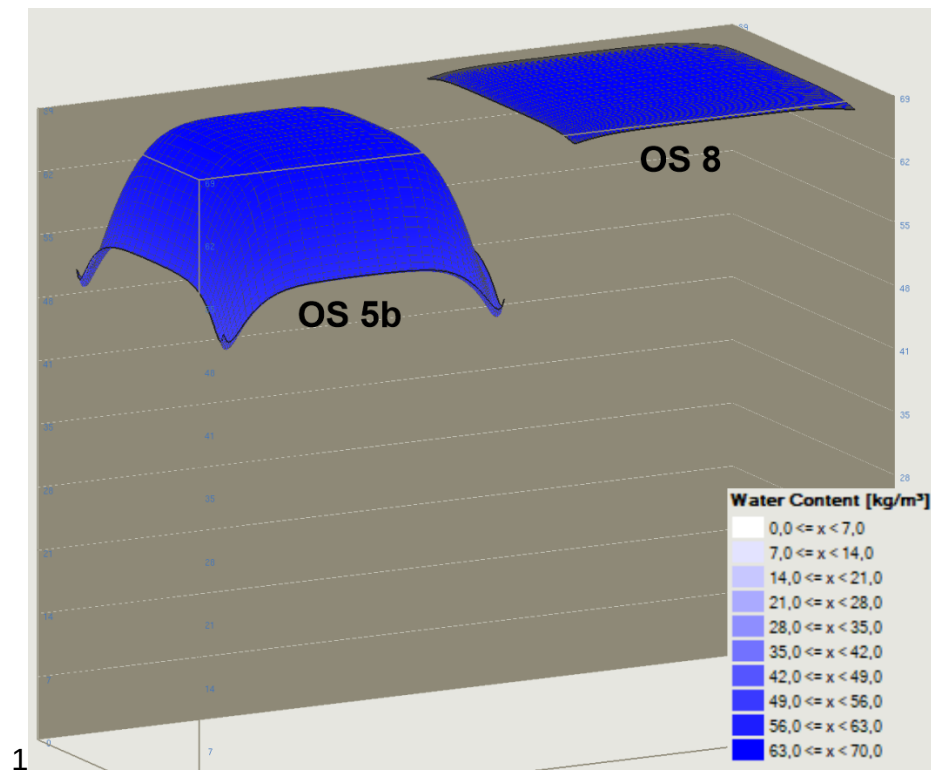


Figure 94: Distribution of water content after 10 years for the two different coatings.

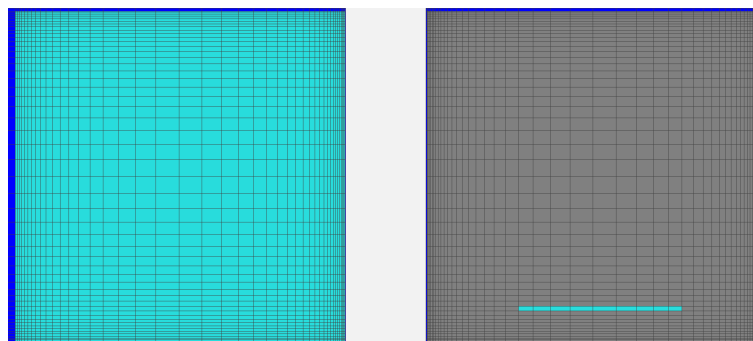


Figure 95: Calculation model with indicated evaluation areas.
Left: cross-section; right: reinforcement layer.

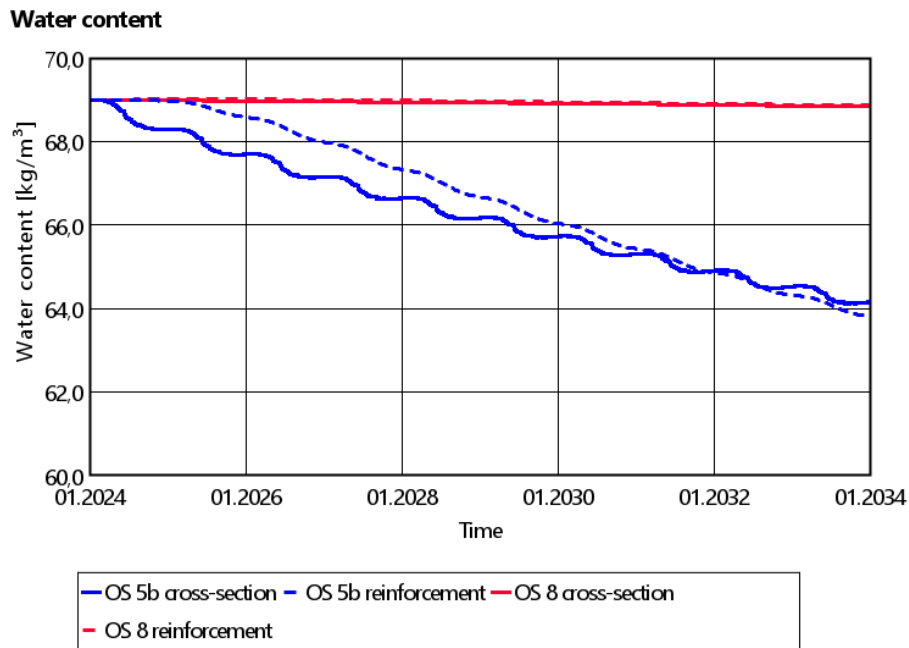


Figure 96: Development of water content in the cross-section and in reinforcement layer in the column for both coatings over time.

Coating concrete components can prevent the absorption of water (rain, splash water) on the one hand, and on the other hand, selecting a sufficient vapor-permeable system may even enable the drying of moist components. Hygrothermal simulations make it possible to predict the drying behavior of components based on the paint / coating used and can therefore help in choosing an appropriate system. With the aid of the new hygrothermal material and corrosion prediction models, it is also possible to predict the progression of corrosion at reinforcement level.

Insulation layers

The following Section examines the effect of retrofitting with exterior insulation as a protective measure to prevent corrosion. Therefore, a double-layer concrete sandwich construction with cavity insulation, in which corrosion was observed on the outer façade, is evaluated (see Figure 97).

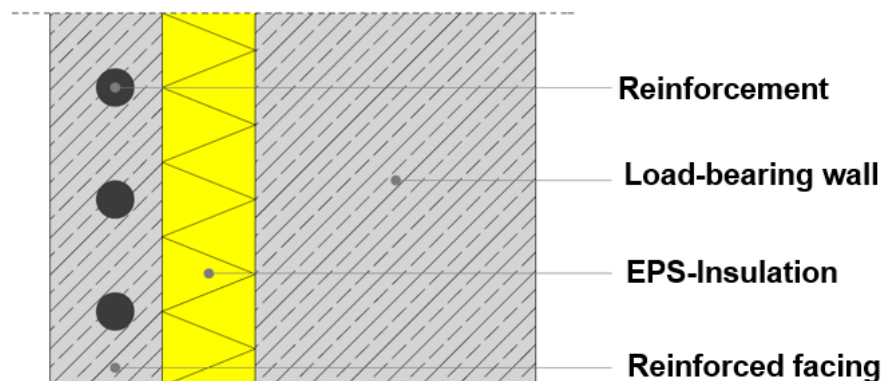


Figure 97: Existing structure: Concrete sandwich wall.

The first step is to calculate the existing wall to determine the cause of the corrosion. Figure 98 shows the temperature and moisture profiles of the simulation for the climate location Holzkirchen over a period of 5 years. Here, the moisture content in the entire inner concrete layer remains below 80 % RH. Corrosion cannot occur in this layer. In the outer, rain-exposed concrete shell and in the reinforcement layer, the moisture content is permanently higher than 80 % RH, so corrosion is to be expected here, if the concrete is carbonized.

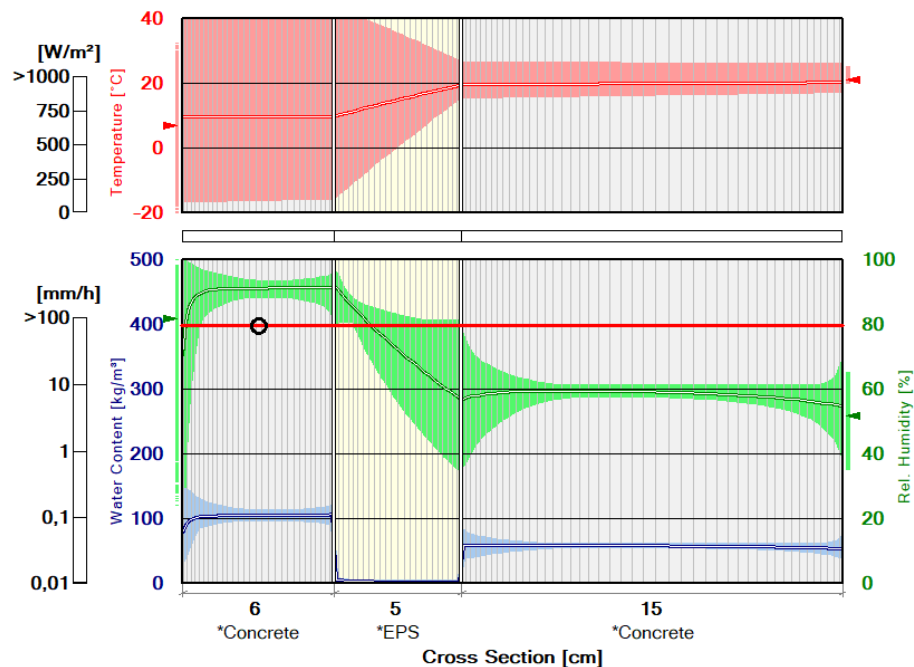


Figure 98: Concrete sandwich wall: profiles of temperature (red), relative humidity (green) and water content (blue) over the calculation period. The lines represent the mean values, and the coloured areas represent the range over the calculation period. The limit value of 80 % RH is shown as a red line. The reinforcement level is marked by a black circle.

To assess the corrosion conditions at reinforcement level, the wall was evaluated using WUFI Corr. In this example, no chlorides are present, therefore the corrosion map for normal concrete is used (see Figure 99). The evaluation shows that corrosion currents exceeding 1 mA/m² (see Figure 100) occur primarily in summer, and material loss of just below 14 µm over 5 years has to be expected. During winter time, when the water in the pores (electrolyte) is frozen, almost no corrosion occurs.

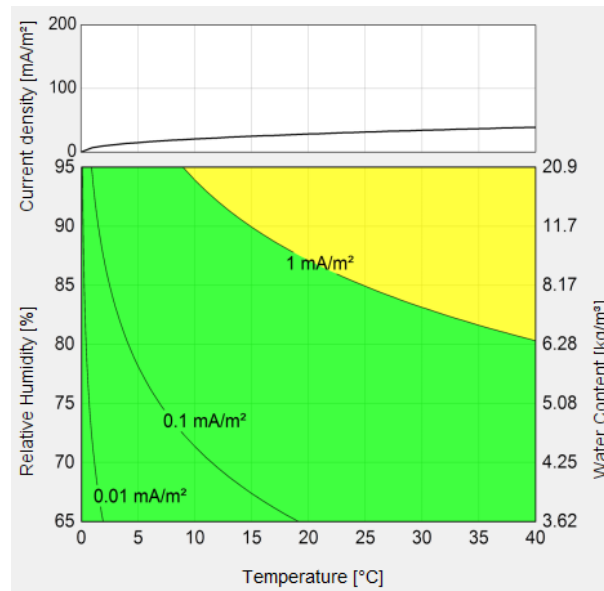


Figure 99: WUFI Corr - corrosion map for the evaluation of concrete without chloride.

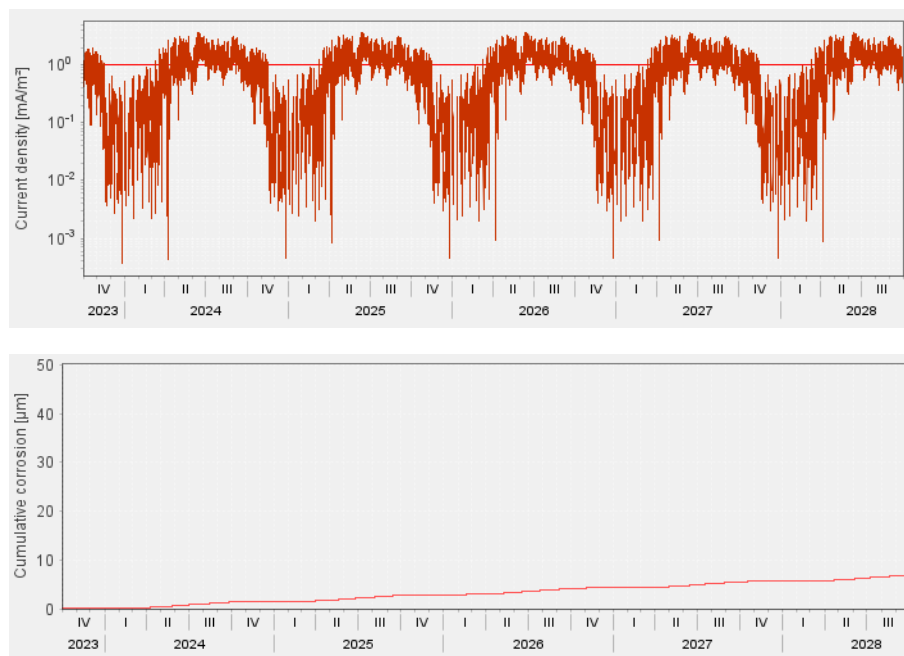


Figure 100: WUFI Corr results:

top: corrosion current flow at reinforcement level.

bottom: total material loss over a period of 5 years.

In the following, the application of an additional insulation layer in the form of a composite thermal insulation system (ETICS) is examined as a renovation measure. The ETICS itself offers a good rainwater protection in the cross-section, as both the plasters meet the relevant requirements and the insulation layer in both variants has a capillary-breaking effect, meaning that rain cannot be transferred from the plaster to the substrate by capillary action. Two systems with the same U-value are compared. Case 1 with 8 cm mineral fibre insulation with a

thermal conductivity of 0.032 W/mK and case 2 with 12 cm EPS insulation with a thermal conductivity of 0.040 W/mK. The construction is shown schematically in Figure 101.

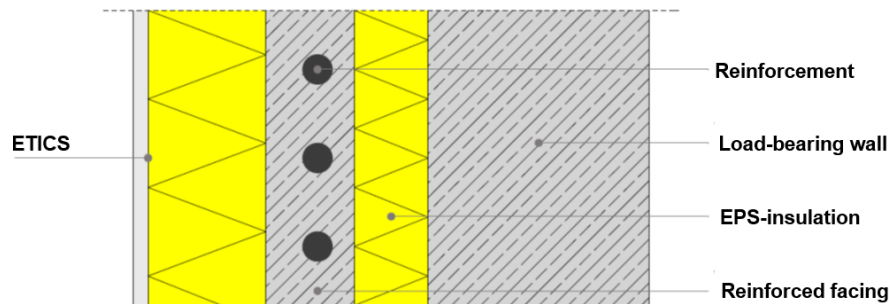


Figure 101: Retrofitting of an existing wall with ETICS.

The water content curve at the reinforcement, calculated with WUFI in Figure 102, shows that both retrofit variants with ETICS (green and purple) can dry out compared to the previous situation (blue). This behaviour is also evident in the relative humidity curves in Figure 103. Here, the moisture at the reinforcement can dry at different rates, again depending on the diffusion resistance of the respective insulation material. Mineral fibre insulation is more vapor-permeable and therefore dries more quickly. With ETICS made of mineral fibre insulation, the corrosion threshold is undershot after about six months, while with the system with EPS insulation, it is undershot after one year.

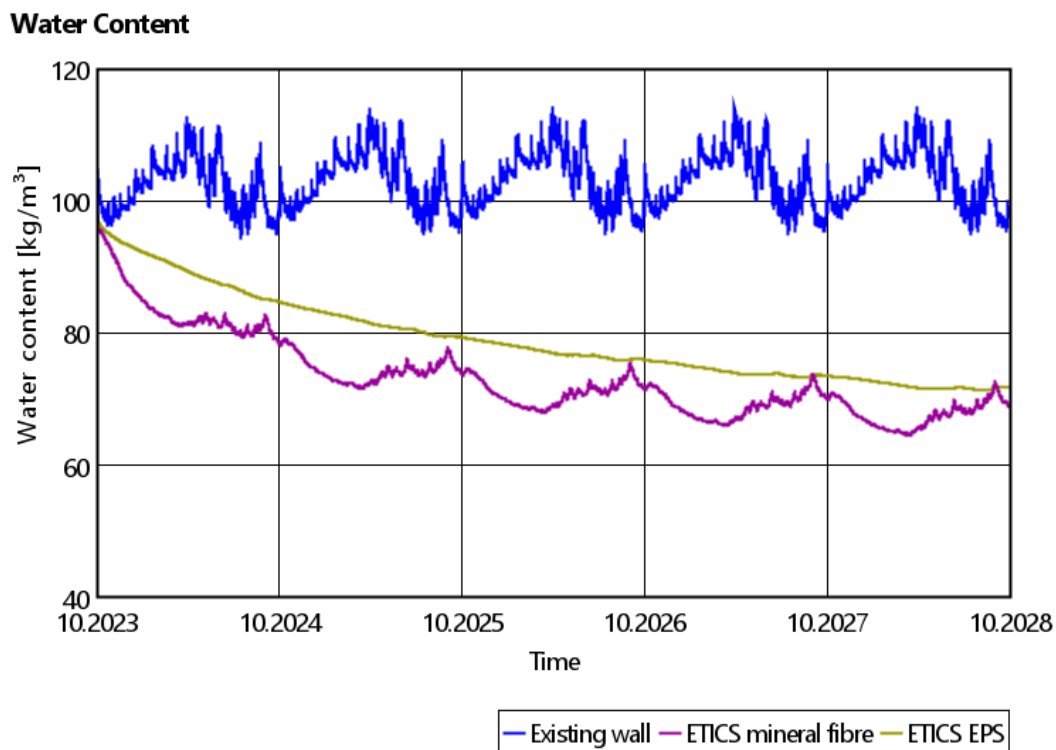


Figure 102: Water content development at the outer reinforcement layer with and without ETICS.

Relative Humidity

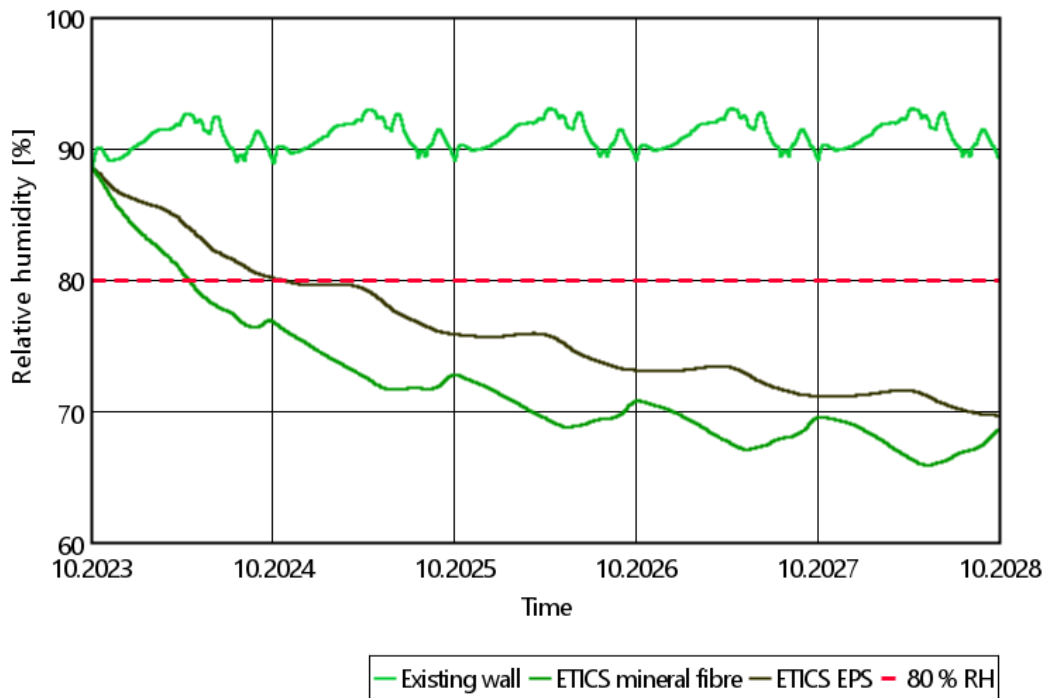


Figure 103: Relative humidity in the outer reinforcement layer without and with ETICS.

The corrosion assessment using WUFI Corr shows a predicted corrosion loss of approx. 1.8 μm (corresponding to approx. 9 μm after 25 years) for the case with mineral fibre ETICS (see Figure 104), while for the case with EPS insulation (Figure 105), the loss is approx. 2.8 μm (corresponding to approx. 14 μm after 25 years, traffic light green). In comparison, the existing variant would achieve a corrosion loss of approx. 7 μm over this period (equivalent to approx. 35 μm after 25 years).

This shows that corrosion in the outer concrete layer can be prevented by applying an ETICS.



Figure 104: WUFI Corr results – ETICS with mineral fibre insulation:
top: Corrosion current flow at reinforcement level.
bottom: Total corrosion loss over a period of 5 years.

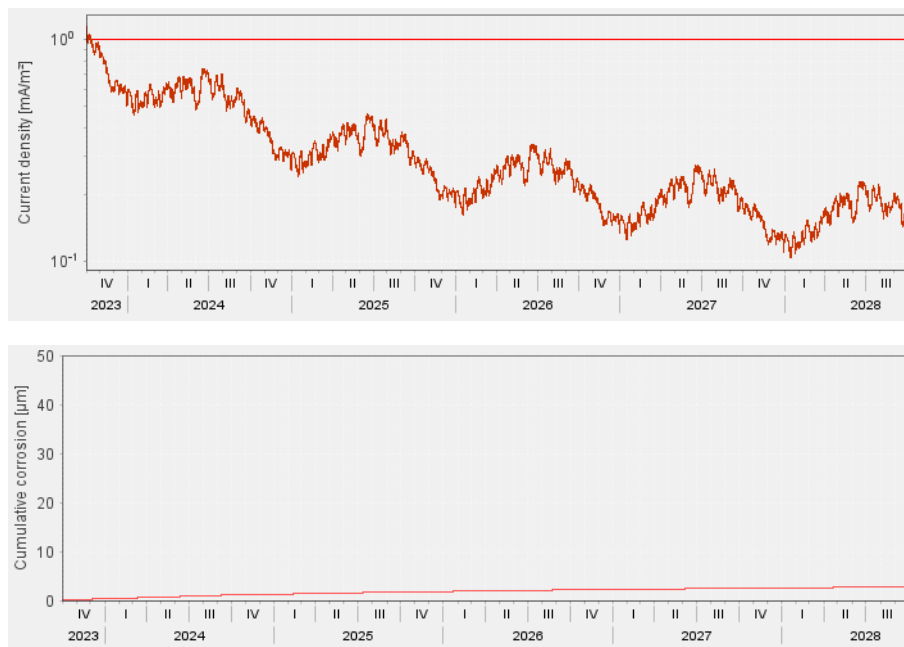


Figure 105: WUFI Corr results – ETICS with EPS insulation:
top: Corrosion current flow at reinforcement level.
bottom: Total corrosion loss over a period of 5 years.

8 Demonstrators and Big-Scale Verification of Simulation Results

8.1 Demonstrator 1: Concrete component with chloride-containing RC concrete

After the storage under corrosion-promoting conditions (20 °C, 98 % RH in a climate chamber) for 108 days, the RC specimens from Chapter 4 were used to set up the corrosion maps for the prediction of corrosion using hygrothermal simulations in Chapter 7. Therefore, the specimen were stored subsequently in an indoor climate (20 °C, 55 % RH) up to an age of a 145 days. The corrosion parameters were measured analogue to 4.3.2. Additionally, the electrolyte Resistance over depth was measured.

8.1.1 Corrosion Tests

Corrosion potential

The corrosion potentials of both anodes (A1 and A2) were measured in accordance with 4.3.2 using a silver/silver chloride (Ag/AgCl) reference electrode. As shown in Figure 106, the observed shift toward less negative corrosion potentials over time e.g., on anode A1, suggests a decreasing probability of corrosion of the RCA-specimens during storage under indoor climate conditions subsequently to storing under corrosion promoting conditions. The specimens cast from a CEMIII with RCAs produced of a concrete with 3 wt.-%/c. chlorides with the most negative potentials in the testing field, exhibited a potential change from approximately -200 mV to -80 mV after a total of 145 days. At a concrete cover of $d_c = 20$ mm Anode A2 of the same specimen showed a change of the corrosion potential from approximately -220 mV to -140 mV.

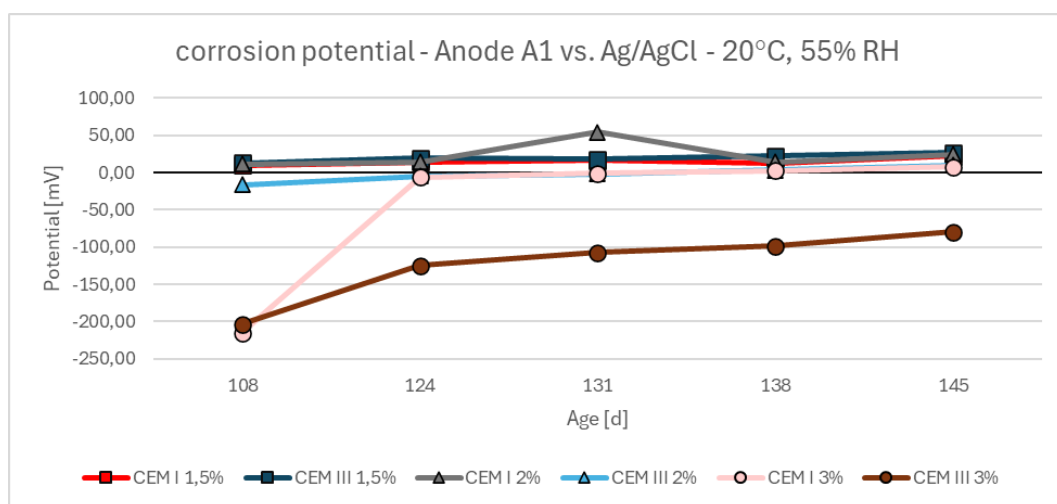


Figure 106: Corrosion Potential of Anode A1 ($d_c = 10$ mm) vs. Silver/Silver Chloride reference electrode versus time (exposure conditions: 20 °C, 55 % RH)

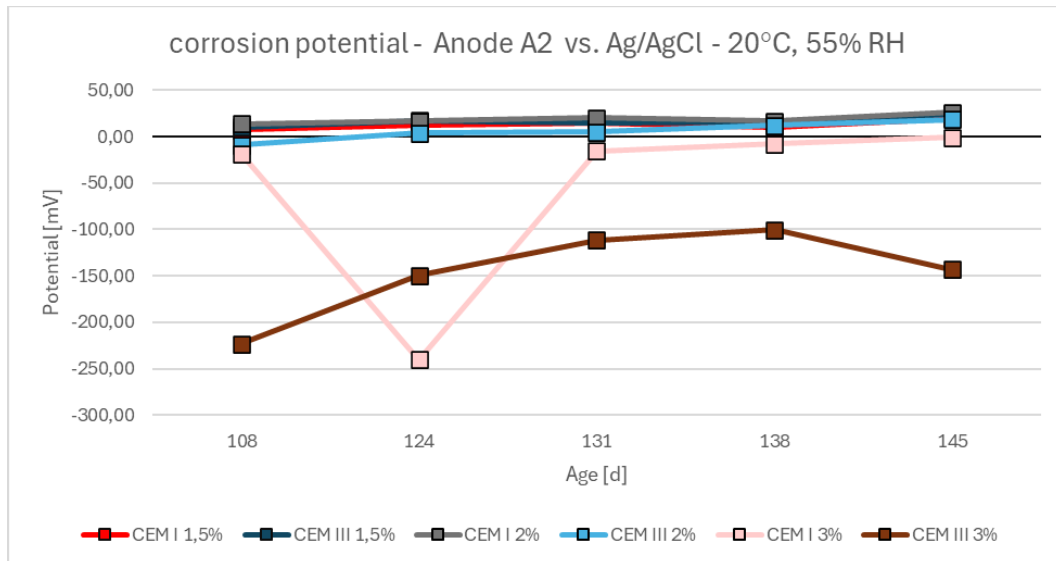


Figure 107: Corrosion Potential of Anode A2 (dc = 20 mm) vs. Silver/Silver Chloride reference electrode versus time (exposure conditions: 20°C, 55 % RH)

Corrosion current density

The investigations of the corrosion parameters, conducted in accordance with 4.3.2, indicate that no detrimental reinforcement corrosion was observed on the anodes during storage under indoor climate conditions (20 °C, 55 % RH) with the exception of specimens made from CEM III with an addition of 3 wt.-%/c. RCAs, and a resulting chloride content of the milled RCAs of 2.14 wt.-%/c. (1,91 wt.-%/c. in the eluate). The element corrosion densities of the CEMIII 3 % specimen showed values from approximately 0,18 to 2.0 $\mu\text{A}/\text{cm}^2$ on Anode A1 and A2, which indicate a high effect of corrosion according to Figure 108.

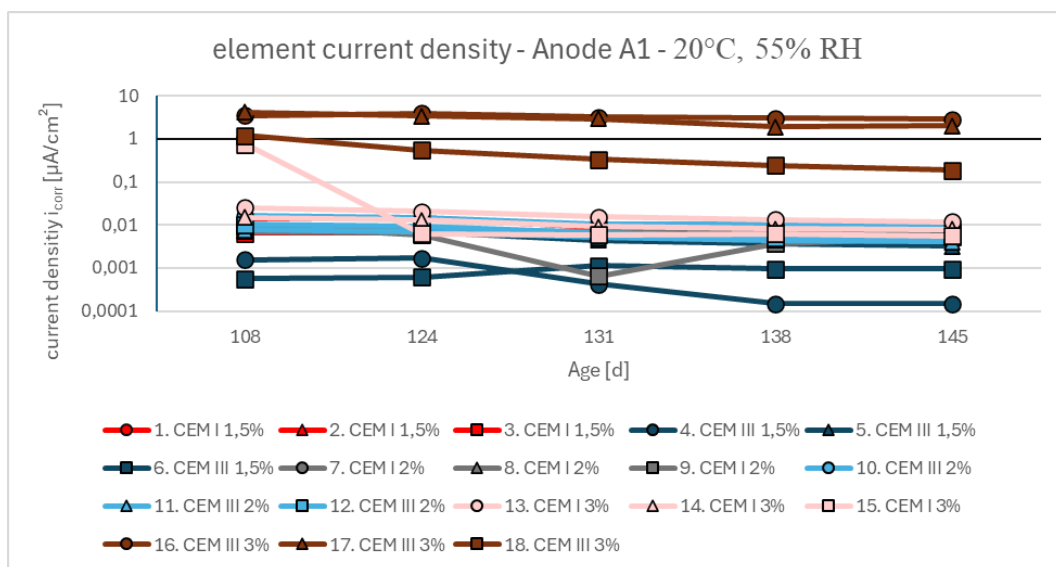


Figure 108: Element Corrosion Current Density of Anode A1 (dc = 10 mm) versus time (exposure conditions: 20 °C, 55 % RH)

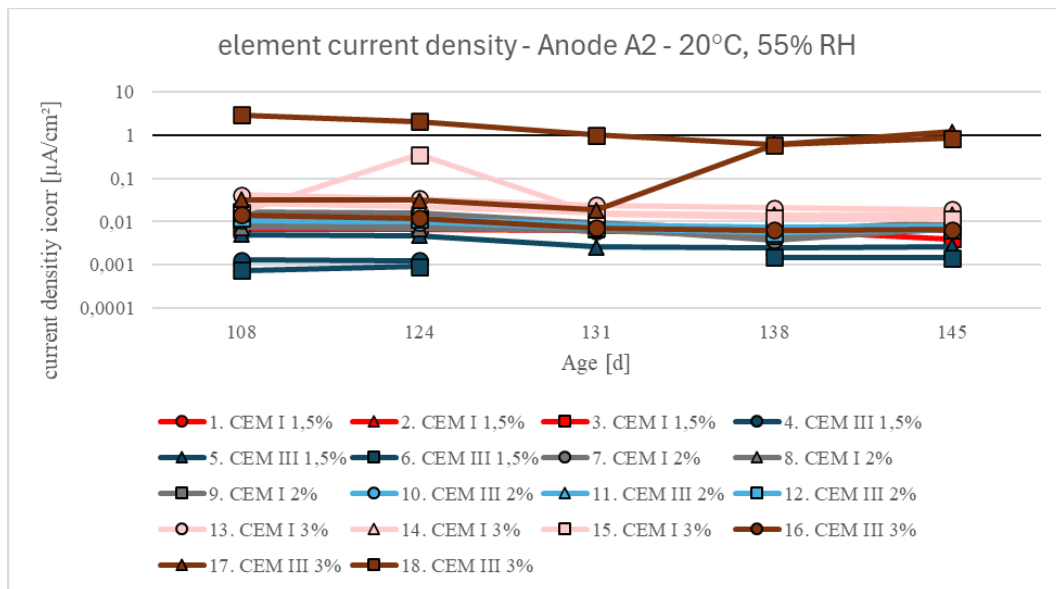


Figure 109: Element Corrosion Current Density of Anode A2 (dc = 20 mm) versus time (exposure conditions: 20 °C, 55 % RH)

Electrolyte Resistance

Based on the embedded V4A electrolyte-ladders as seen in Figure 39, the electrolyte resistances could be measured in 5 mm increments down to a depth of 45 mm. The following diagrams illustrate the development of electrolyte resistance over the storage period under indoor climate conditions (20 °C, 55 % relative humidity).

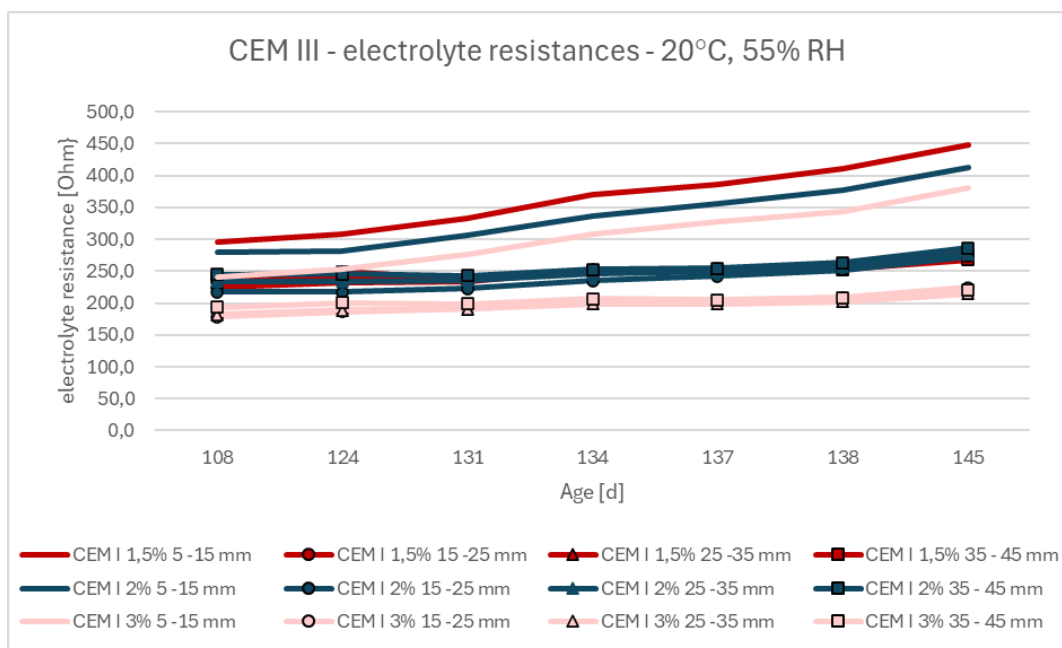


Figure 110: Electrolyte resistances in Ohm over time (20 °C, 55 % RH) of the concretes produced with CEMI

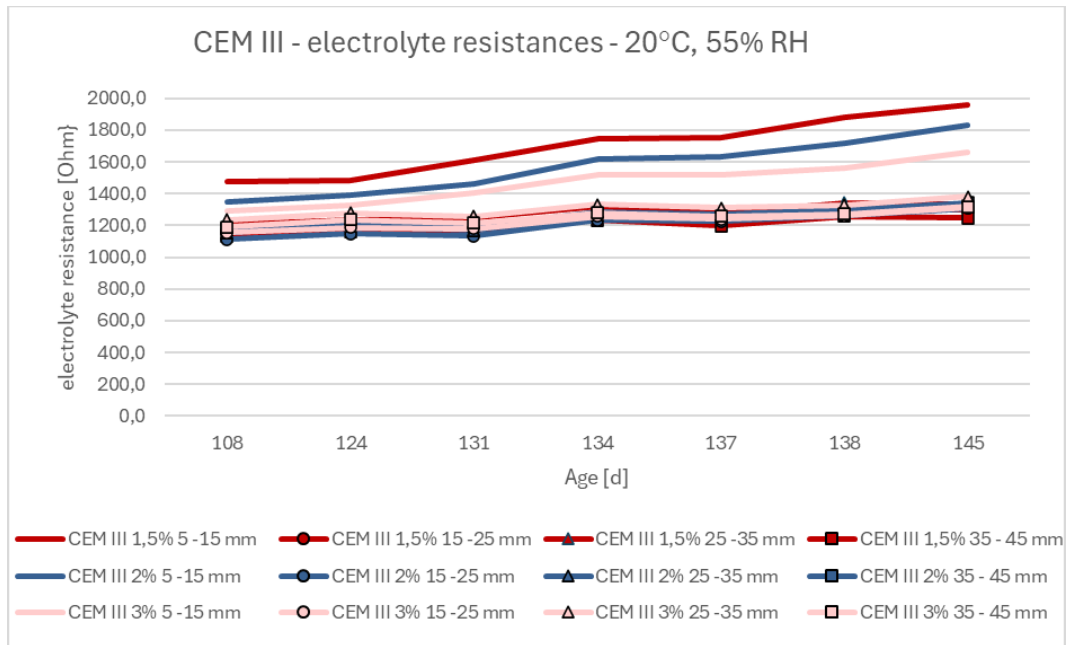


Figure 111: Electrolyte resistances in Ohm over time (20 °C, 55 % RH) of the concretes produced with CEMIII

As seen in Figure 110 and Figure 111 the electrolyte resistances increased slightly across all measured depths, with specimens containing the lowest chloride contents exhibiting the highest resistances. This can be attributed to the fact that the electrical conductivity of chlorides reduces the overall electrolyte resistance in concrete at higher chloride concentrations. In accordance with the cement types used, concretes with blast furnace cement (CEM III) showed higher resistances compared to those with Portland cement (CEM I).

8.1.2 Additional Corrosion Tests

In Addition to Demonstrator 1 prismatic specimens with dimensions of 278 mm × 200 mm × 135 mm (10.94 in × 7.87 in × 5.31 in, L × W × H) were prepared. Except in the reference series (no chloride addition) all specimens were equipped with a manganese dioxide reference electrode comprising four Anodes (B500 steel rebar sections welded to V4A stainless steel wires with concrete covers of 20; 40; 60; and 80 mm). The cathodes were made of titanium mixed metal oxide (TMMO) strips. The surface area ratio of anodes to cathode was 1:15. A schematic illustration of the corrosion specimen setup is provided in Figure 112.

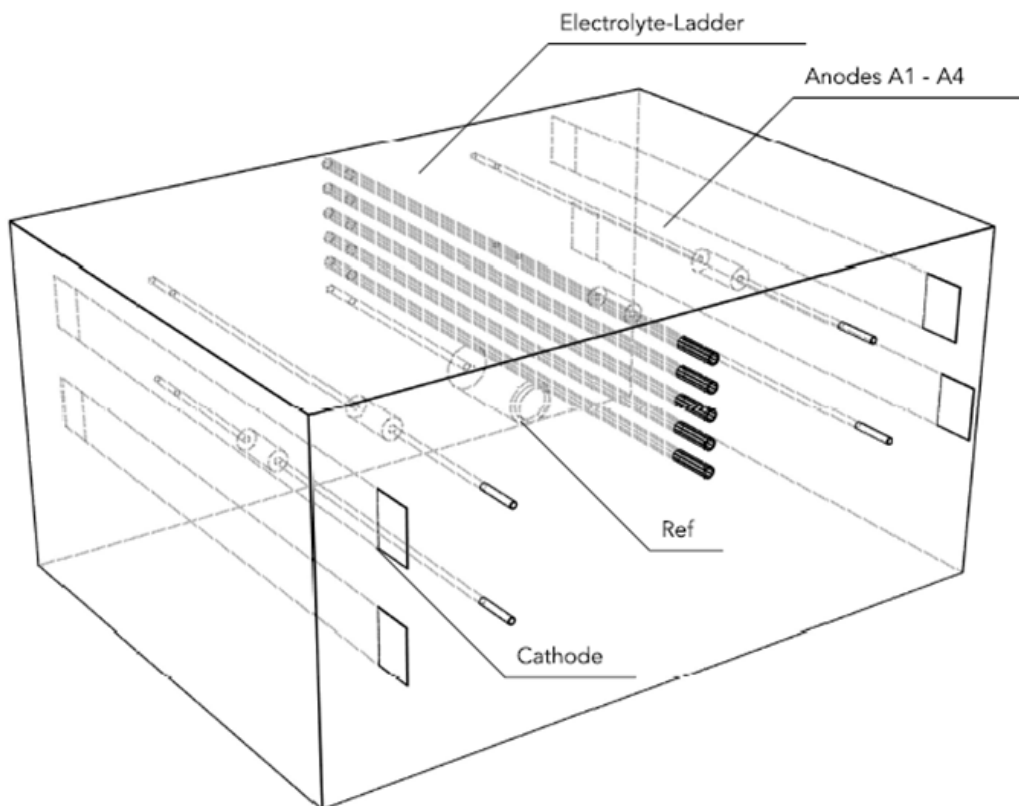


Figure 112: Specimen with three-electrode setup for the corrosion investigations

Concrete

The Concrete used for the corrosion test specimens were produced with a maximum particle size of 8 mm (0.31 in, A/B8), combined with the same types of cement as in the original parent mixes of WP 3.4 (CEM I 42.5 N and CEM III/A 42.5 N). A constant water-to-cement ratio of 0.45 was maintained. The composition of the concretes is presented in Table 36.

Table 36: Composition of the Concretes (CEM I 42,5 N and CEM III/A 42,5 N) for the production of the corrosion specimens

Material	Mass	Density	Volume
[-]	[kg/m ³]	[kg/dm ³]	[dm ³]
CEM	350	3.1	113
Water	193	1	193
0/4	1224	2.67	458
4/8	589	2.67	221
Air voids	-	-	15
Total	2356	2.356	1000

Corrosion Current Density

Based on the specimen configuration, element currents between the anodes A1 – A4 ($d_c = 20 \text{ mm} - 80 \text{ mm}$) and the cathodes opposite to each cathode can be recorded stationarily using a datalogger (Ahlborn Almemo 710).

The specimens were stored in a climate chamber under varying environmental conditions. The temperature was also documented via a data logger. The specimen designation is based on the binder type (CEM I / CEM III) and the chloride addition amount used during the production of the concretes (0.0 wt.-%/c., 1.5 wt.-%/c (CEM I only), 2.0 wt.-%/c, 3.0 wt.-%/c) and the measured Anode (A1 – A4). The results of the measurements are shown in the following Figure 113.

the determined relative corrosion rates of the test specimens at the measured 39.4 °C are recorded. There is a very large scatter of the relative corrosion rates, with an average that can be well represented by $a = 7,500$ K. Furthermore, it is evident that high chloride levels in the test specimens also lead to high relative corrosion rates, which is equivalent to a high α -value.

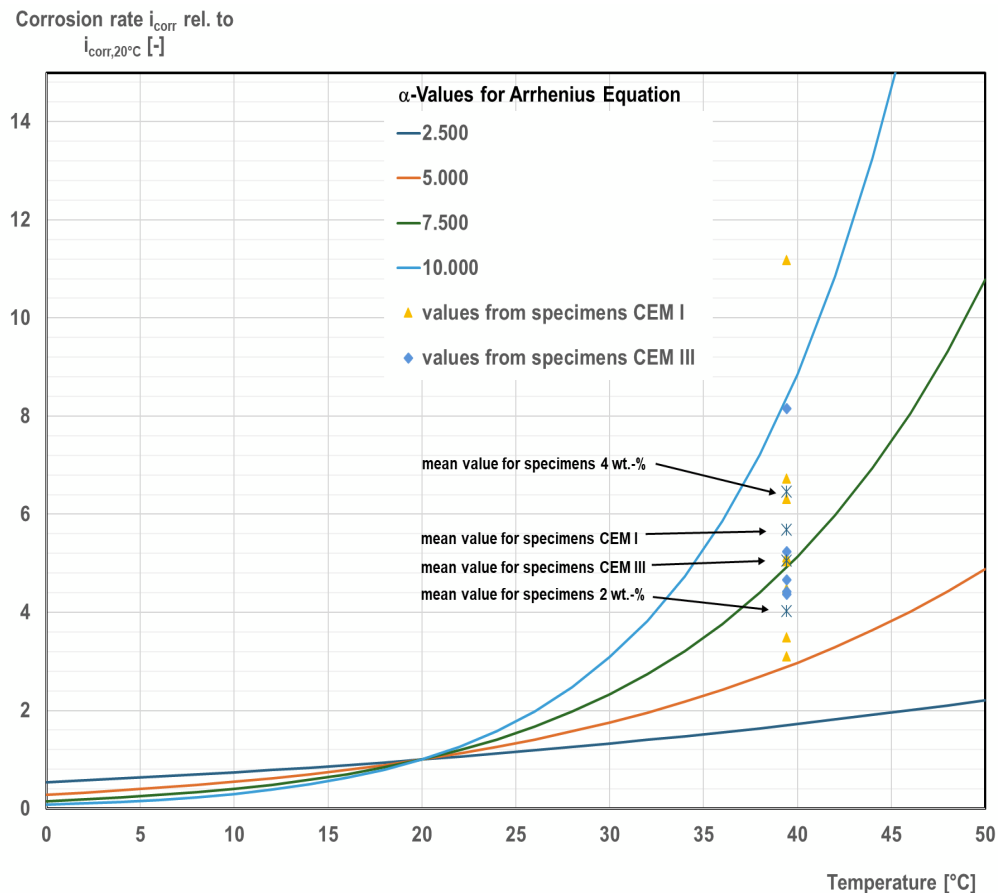


Figure 114: Relative corrosion rate at temperature T with reference temperature of 20°C according to Arrhenius with values from specimens

8.2 Demonstrator 2: CO₂ capture and storage of RC concrete

The main focus of this study was to produce demonstrators to assess the influence of forced carbonation of recycled aggregate used in RC concrete mix designs on the carbonation behaviour of cube samples.

The RC-samples described in Table 14 were produced with naturally carbonated recycled aggregates and forced carbonated recycle, as shown in the Figures below. The sample preparation, storage and determination of the carbonation resistance are carried out in accordance with (OENORM EN 12390-12 2020). The exposure time in the accelerated carbonation process was predefined with 28 days.

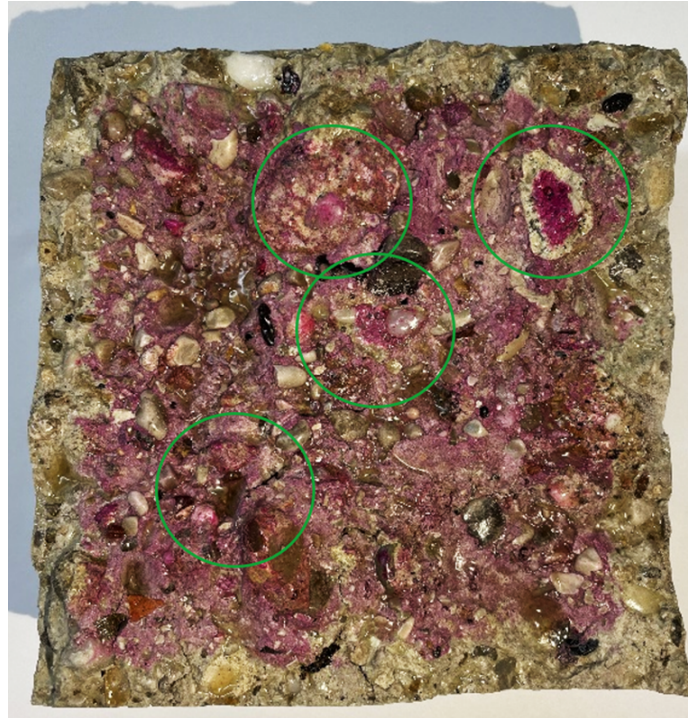


Figure 115: Split cube specimen of recycled concrete mix type C-2 containing 38 % recycled aggregates

Figure 115 distinctly shows the recycled aggregate embedded in the concrete with natural carbonation progress due to the freely weathered storage after the recycling process. The average carbonation depth of 3 test specimens (C-2, RC not carbonated) was determined to be 11.6 mm

Particularly in the case of larger grain fractions, the recycled aggregates become visible with their non-carbonated areas through a phenolphthalein test. Due to the embedding of non-carbonated recyclates in new cement paste or the location inside the cross-section, no additional carbonation can take place in normal cases. The storage potential therefore remains unused. The use of incompletely carbonated recyclates for the production of recycled concretes therefore leads to a fundamentally high-quality reuse of demolition concretes, but the CO₂ storage potentials are not used acutely and remain largely unusable until the new recycling process.

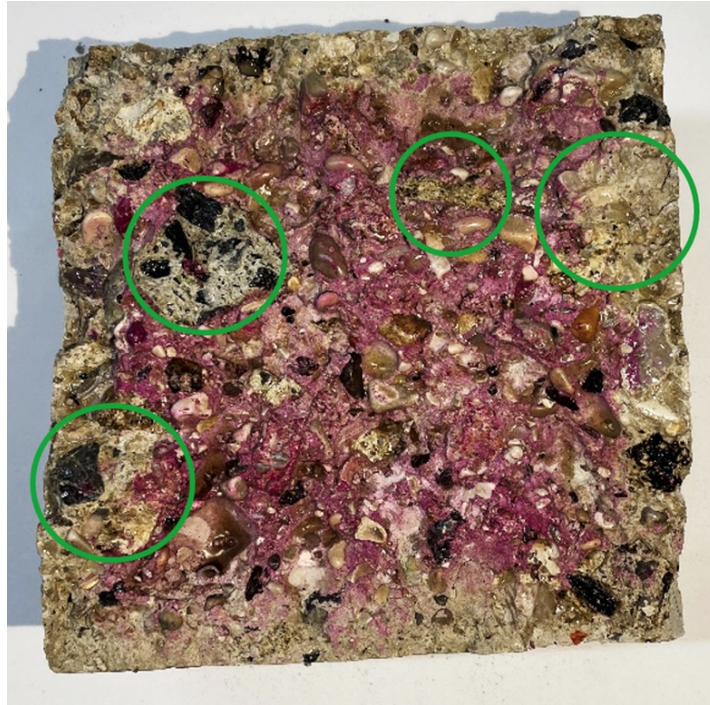


Figure 116: Split cube specimen of recycled concrete mix type C-2 containing 38 % recycled, forced carbonated aggregates

Figure 116 shows the fracture surface of the split samples (C-2, recycled aggregate fully carbonated) with the visibly enclosed, CO_2 -saturated recycled grains, which no longer show any coloration when the phenolphthalein solution is sprayed on due to the neutral pH value. The average carbonation depth of 3 test specimens (C-2, RC fully carbonated) was determined to be 13.1 mm

As a reference to the recycled concrete mix design, carbonation tests were carried out on the standard mix design C-0. Figure 117 shows an example of a phenolphthalein test on a split sample (C-0) from a series of 3 pieces with an average carbonation depth of 12.4 mm.



Figure 117: Split cube specimen of standard mix design C-0

The comparison of the determined carbonation depths Table 37 shows the usual deviations of the individual samples. No special tendency of the material behavior depending on the aggregate used can be derived.

Table 37: Mean values of the carbonation depths of the specified sample types after 28 days of CO₂ exposure

Sample	C-0	C-2, recycled aggregates not carbonated	C-2, recycled aggregates fully carbonated
Carbonation depth [mm], Mean value from 3 test specimens of each type with 4 data points each	12,4	11,6	13,1

9 Handling instructions for application areas of RC concrete containing chloride

Concerning the **Concrete technology** the use of chloride containing recycled aggregates is possible for concrete with exposure classes XC1 und XF1 up to a chloride content of 3 M.-%/cement content. Fresh and hardened concrete properties were nearly not influenced by using recycled or chloride containing recycled aggregate. The Young's Modulus of 100 % RC is most time about up to 25 % lower than with natural aggregates. The shrinkage is higher due to the high amount of cement stone in the 100 % RC mixture. Both properties are not influenced by the chloride content in the RA. The chloride content in the investigated RA was always about 35 % higher than the chloride content in the produces RC with 100 % chloride containing RA. For the use of chloride containing RC should be taken into account, that an elution of chlorides is to be excluded. All the results in this research report are tested up to an age of about 120 days. Long-term experience should be investigated further.

In Figure 118 the possible applications are summarized.

Recommendations for applications of recycling concrete with 100% recycled aggregate containing chlorides according to CRUFI results

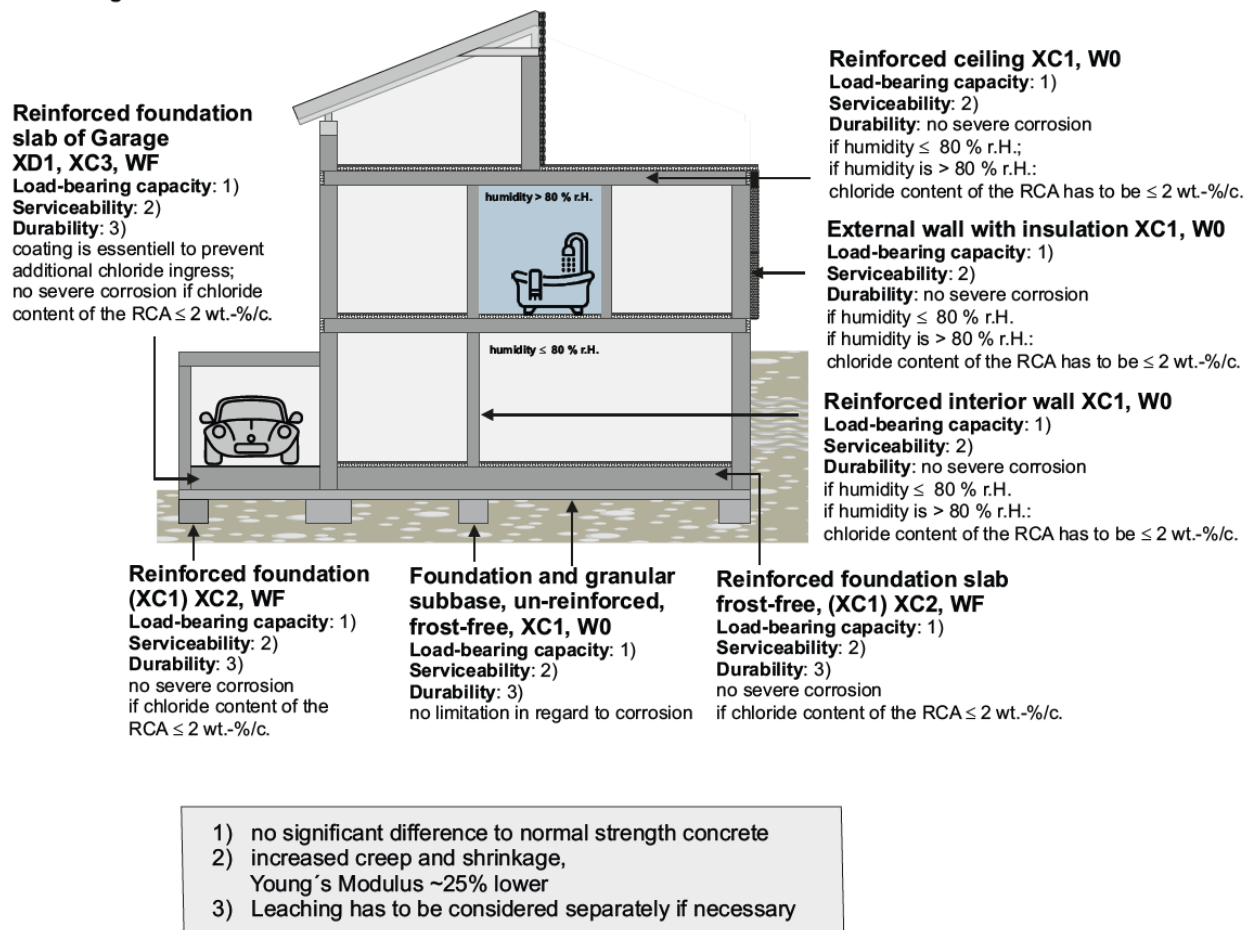


Figure 118: Recommendations for applications of recycling concrete with 100 % recycled aggregate containing chlorides according to CRUFI results

Regarding the **risk of corrosion** for the reinforcement, it can be derived from the results of the investigations that for components that are permanently stored in a dry environment and where there is no risk of condensation forming in the cross-section, harmful corrosion is not to be expected. A relative humidity of 80 % is defined as the boundary of the dry environment. The results refer to investigations of up to 3 wt.-%/c chloride content in RCA. Therefore, chloride-containing RC concrete can also be used for typical internal components.

In case of higher air humidity (wet rooms) or with expected formation of condensate, or for example when there is water from the underground at the components, the chloride content in the RCA must be limited to ≤ 2 wt.-%/c. Here, the results show no harmful corrosion.

It should be noted, however, that bound chlorides can be released by carbonation of the RC concrete, which makes damaging corrosion possible even at lower chloride levels. Therefore, it should be taken into account that during the service life of the structure, the carbonation front does not reach the reinforcement.

From the **hygrothermal perspective**, it is possible to check by simulations whether the boundary and climatic conditions lead to hygrothermal conditions within the structure and especially at the position of the steel remain in a favorable range for the long-term use for RC concrete containing chlorides.

In general, components that are not expected to become humid due to the climatic loads, such as interior walls or components without reinforcement, are uncritical. If the climate conditions in the room remain below 80 % relative humidity in the long term, which is normally the case in all buildings with residential or office operation, no critical conditions are to be expected. Care has to be taken in case of interrupted operation – but conditions above 80 % TH will harm the assemblies anyway – also concerning other damages like rot or mould growth and should therefore be avoided to protect the fabric. Interstitial condensation in the components, which can occur due to temperature gradients and differing moisture conditions must be also safely ruled out. Such situations can occur, for example, in case of interior insulations made without vapor retarders. Exterior walls of buildings and components such as concrete columns must be protected appropriately from rain and splash water.

Hygrothermal simulations can be used to check whether the limits for application are being complied with. The simulations enable a reliable prediction of the temperature and moisture development in the component and especially in the layer of the reinforcement.

10 Summary and conclusion

A total of seven different demolition materials were compared. Six of them originated from concrete obtained from demolished German infrastructure structures, while one was derived from laboratory specimens made from defined concrete mixtures. The aim was to investigate whether a limited chloride concentration in RCAs is tolerable, particularly for components with low performance requirements, without initiating reinforcement corrosion or while keeping corrosion activity at a low level.

In addition, five different RCAs were investigated in Brazil and Austria and described in detail. All mixtures fulfilled the requirements.

The results show that RC produced with chloride-containing aggregates from demolished structures exhibited a lower chloride content, both in the crushed material and in the eluate of the hardened RC. This means that the chloride content could be reduced by up to 35 % by using it in the concrete mixture. In contrast, the RC produced with crushed laboratory concrete still showed a higher chloride content in the eluate of the hardened RC after mixing. It seems, that the binding behavior of the chlorides may be different from that of the RCA (TS, CC, OC, LS, TK and GG) samples from demolished built structures. The chlorides may therefore be present in free form in the laboratory (Cl-L)-samples.

Three preferred concrete mixtures were produced for exposure classes XC1 and XF1. A compressive strength of C16/20 was required for XC1 and C25/30 for XF1. The concrete mixes were produced both with and without limestone powder as an additive. Additionally, a reference mix without chloride-containing RCA was produced. The durability performance of RC containing chloride-bearing aggregates was assessed on the basis of the time-dependent development of compressive strength, freeze–thaw resistance, and carbonation behavior. The freeze-thaw resistance criteria were fulfilled for XF1 for the RC containing chlorides in the RCA. The RC with chloride containing RCA containing limestone powder showed higher carbonation depth than the other RC mixtures.

Five different concrete mixtures with different amounts (0 % up to 100 %) of RCA were examined. In addition, based on the available recycled aggregates and the limits specified in ÖNORM, four different partial replacement rates were defined for the experimental program.

The potential replacement of OPC with alternative binders was also investigated. The use of fine limestone powder and RCA fines was determined. The limestone powder was tested in combination with the chloride containing RCA and durability tests.

Three different chloride contents with a CEM I and CEM III were used to produce parent concretes for the RCAs. The RCAs were stored in a climate chamber for 28 days followed by approximately one year of storage under laboratory conditions. The concretes were then crushed to produce the RCAs. For the RCs the RCAs were used again with a CEM I and CEM III and then stored under corrosion promoting environmental conditions (20 °C, 98 % RH). Electrochemical measurements (ZRA) of corrosion potentials and corrosion current densities were conducted over a period of 108 days. The investigations regarding the corrosion parameters show that, with the exception of specimens made from CEM III and those

with an addition of 3 wt.-%/c. for the production of the RCAs, and a resulting chloride content of the milled RCAs of 2.14 wt.-%/c. (1,91 wt.-%/c. in the eluate), no detrimental reinforcement corrosion was observed on the anodes due to the recorded corrosion potentials and low corrosion current densities over time.

The tests carried out demonstrated reproducibility of the results for both the concrete aggregate samples and the cement-based specimens, taking the calibration tests into account. This applies to both the recycled aggregates tested and the pure cement test specimens. It should be noted that in the case of forced carbon dioxide exposure of recycled aggregates, satisfactory results can be achieved with coarse grain fractions in a static carbonation process. If fine aggregates or aggregate mixtures containing sand fractions are to be used for CO₂ storage, sufficient airflow through the material cannot be archived to fully exploit the available storage potential. However, recycled sands in particular contain a high proportion of reactive cement paste, which means that there is great potential for ecological optimization in terms of carbon storage.

In addition, the volume-to-surface ratio is naturally an essential basic parameter for a possible forced carbonation process. A static process for the treatment of grains can only be assessed as suitable to a limited extent, as large areas are only exposed to insufficient gas exchange. For further experimental model tests, a process is to be investigated in which it is possible to move the bulk material during the carbonation process.

Especially in connection with the conservation of primary sand and gravel resources, the high-quality reuse of demolition materials and the possibility of storing carbon dioxide in the recylate, a significant potential of forced carbonation for the ecological compatibility of the concrete production process can be demonstrated. This study demonstrates the high potential for the use of recycled concretes in massive infrastructure structures and buildings and the possibility of accelerated long-term storage of CO₂ in concrete construction.

Material datasets for different concretes without and with chlorides were measured for use in hygrothermal simulation tools. This makes it possible to calculate the long-term behavior of these materials regarding the temperature and moisture situation in constructions as well as the hygrothermal conditions in the reinforcement layer, which is used later to predict the corrosion risk depending on the embedding material.

A new concrete material model was developed which allows a realistic representation of the moisture redistribution in the concrete components. Accompanying field tests were carried out on concrete prisms, enabling validation of the models.

For building components protected from rain or other sources of water, the typical built-in moisture present after the main phase of hydration (approximately four weeks after casting) is close to the equilibrium moisture content at 95 % RH. If chlorides are involved, they also bind a significant amount of water from the ambient air, which increases the equilibrium moisture content above the deliquescent moisture of the salt (75 % for NaCl). The increase depends also on the type of salt and can vary in practice depending on the salt distribution. This means that conservative worst-case assumptions should be used for design purposes. For the tested specimens the increase in water content above 95 % RH ranges from 30 to 60 %

higher equilibrium moisture contents. The test specimens in this project were produced by mixing a defined amount of chlorides directly with the cement and aggregates during production. This leads to a homogeneous distribution of salt in the cement matrix. In practice, however, this salt originates from the environment and is therefore concentrated in the pore system of the material. Reproducing this type of salt distribution in the laboratory is hardly feasible or would require many years; it was therefore not an option within the scope of the project. It is likely that the two different salt loading procedures also lead to different influences on the material properties. Therefore, further research on this topic is required in future.

Due to incomplete measurements of corrosion behavior, only empirical evaluations and modelling could be carried out. The measured results vary greatly and the final water content profiles for the test specimens were not yet available. Therefore, the resulting corrosion maps remain uncertain and could not yet be reliably validated.

A detailed evaluation of the conditions of the corrosion tests is still required. Since the measurements continue to show significant variations, partly due to the material, it would be useful to carry out further measurement series. Based on these, further specific corrosion maps should be created.

Within this framework, the effect of coatings, paints and insulation on the moisture behavior of concrete was also investigated. It was shown that such measures are suitable for preventing moisture absorption in the concrete and at the same time enabling it to dry out. The aim here is to dry the concrete at the reinforcement layer in damp areas so that corrosion is limited. It was demonstrated that hygrothermal simulations are the method of choice for reliably assessing the long-term moisture behavior of the structure and reinforcement layer and for using them in conjunction with corrosion current measurements to predict the effectiveness of a measure.

Demonstrator specimens were produced and stored under indoor climate conditions (20°C, 55 % RH) and electrochemical measurements (ZRA) were conducted. In addition, electrolyte resistances were measured and recorded. In addition, a second demonstrator was produced with the same chloride contents and with CEM I and CEM III cements. The specimens were stored in a climate-controlled chamber under different temperatures and humidities. The measurements were conducted under short-circuit conditions and are still ongoing. A gravimetric determination of the moisture content will be carried out upon completion of the measurements. Based on the data obtained so far, a correlation between temperature and corrosion rate could be established.

Concrete cube test specimens with replacement rates of natural aggregate by concrete recycle of 38 % were produced. One series of specimens was produced with incompletely, naturally carbonated recycle, the other series with completely, forced carbonated recycle. Subsequently, the carbonation resistance of the samples was determined in a standardized test according to EN 12390-12. The test results were compared with known test data of standard concretes, whereby no significant difference in material behaviour was found.

Likewise, no negative effects of the accelerated-carbonated material on overall material behavior could be identified. This applies to both the strength and durability parameters of the tested samples.

Handling instructions have been formulated, indicating the components for which RC concrete with chlorides of up to 3 % can be used.

11 Outlook

In future research, a broader range of demolition materials from a wide variety of demolition sites should be investigated in order to better assess the actual chloride contents present in existing infrastructure and the corresponding variability. It should also be examined to what extent the concrete mix design influences the reduction of chloride content in the recycled concrete. In addition, further exposure classes should be considered.

Detailed investigations of chloride binding behavior are also required, as the laboratory samples with added chlorides consistently behaved significantly differently from the samples obtained from real structures. This raises the question of how chlorides should be introduced into laboratory mixtures in order to realistically reproduce field conditions.

Further material datasets for concrete, together with corresponding corrosion tests for different concrete compositions, should be generated in order to validate the existing corrosion maps and develop additional ones, thereby enabling corrosion prediction for a wider range of applications and boundary conditions.

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