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Contents

1 Introduction ... 4
2 Litterature review/Background ... 5
2.1 Ultra High Performance Concrete ... 5
2.2 Use of UHPC in transportation infrastructure ... 5
2.3 Requirements and standards ... 6
2.4 Standards ... 8
2.5 Repair techniques for concrete structures ... 9
3 Material development ... 12
3.1 Material ... 12
3.2 Mixture development ... 12
3.2.1 Restrictions and criteria ... 12
3.2.2 Mixdesign ... 12
3.3 Final mixtures ... 14
3.3.1 Mix design ... 14
3.3.2 Compressive & flexural strength ... 15
3.3.3 Shrinkage ... 16
3.3.4 Durability ... 18
3.4 UHPC as an overlay ... 20
3.4.1 Reference concrete ... 20
3.4.2 Casting ... 20
3.4.3 Adhesions tests ... 22
3.4.4 Bending ... 25
3.4.5 Field test ... 26
4 Large scale testing ... 27
4.1 Test setup ... 27
4.2 Mechanical properties ... 28
4.3 Bond strength ... 29
4.4 Crack depth ... 30
4.5 Preliminary finite element model ... 31
5 Life Cycle Assesmen/Life Cycle Costs ... 35
5.1 LCA ... 37
5.1.1 Background data ... 37
5.1.2 Impact assessment ... 37
5.1.3 GWP of production and transport of 1 m^2 of concrete ... 39

5.1.4 GWP of repair 39
5.2 LCC 40
5.2.1 Per cubic meter 40
5.2.2 Costs for concrete cover 41
5.2.3 Considering repair costs 42
6 Conclusion/discussion 43
6.1 UHPC recipes properties 43
6.2 LCA/LCC 43
6.3 Future research 43
7 REFERENCES 44

1 INTRODUCTION

The transportation infrastructure is exposed to some of the harshest conditions for concrete, including carbonation, frost and chloride attacks. In addition, structures can be damaged by accidents or overloading. The durability and high maintenance costs of concrete and steel bridges have become a major challenge in many industrialized countries (*Gjorv 2013, Tang et al. 2012, ACI 2006*).

One of the novel trends, rapidly spreading throughout the world, is the usage of Ultra High Performance Concrete (UHPC). Mostly due to its high strength and extremely low permeability, enabling manufacturing of lightweight and durable elements. The UHPC can be used for new structures as well as repairs and strengthening of existing, deteriorated elements and structures.

A major drawback of traditional UHPC is the high cement content. This project aimed to create an ecological UHCP used for repair of concrete bridge pillars in harsh environments than won't require almost none if any maintenance. By using cementitious materials such as fly ash and lime stone, more environmentally friendly UHPC have been developed with both very good casting, structural and durability properties.

2 LITTERATURE REVIEW/BACKGROUND

2.1 Ultra High Performance Concrete

The UHPC could be shortly characterized as extremely well optimized normal concrete. The UHPC is based on Portland cement but has a very low water to binder ratio (w/b), large additions of silica fume (SF) and well packed particles. The 28-day compressive strength can reach up to 250 MPa and the flexural strength up to 35 MPa (*Russel and Graybeal 2013, Graybeal 2011 and Cwirzen et al. 2008*). Enhanced mechanical properties enable to decrease the size of the produced elements but at the same time also to sustain the same load bearing capacity, Figure 1-1. The microstructure of UHPC concrete is extremely dense which hinders penetration of any aggressive media and thus stop/prevents most of the deterioration processes.

Figure 1-1 Comparison between elements having approximately the same load bearing capacity: (left) steel, UHPC and NC and (right) NC and UHPC.

The basic ingredients for UHPC concrete are very similar to normal concrete (NC) and especially to HPC concrete. Ordinary UHPC include Portland cement, fine sand, silica fume, superplasticizers, water and sometimes various types of fibers.

UHPC concrete can be mixed using conventional pan or drum mixers (*Graybeal 2011*). One of the biggest problems related to mixing of UHPC concrete is its large fine content and lack of coarse aggregates which in connection with extremely low water content and required large dosage of superplasticizer lowers the efficiently of mixing. As a result the mixing time is elongated from 1-2 minutes to 10-18 minutes (*Mazanec at al. 2010*).

The mixing time can be also reduced by a better mix design, optimized particle packing, and choice of right chemical admixture or by adjusting the mixing sequence (*Mazanec at al. 2010*).

2.2 Use of UHPC in transportation infrastructure

Bridges and other transportation infrastructures are mostly build using normal and to a limited extend also high strength concrete (28-day compressive strength at >60 MPa). Precast concrete has been used for bridge construction for many decades. One of the drawbacks of a normal concrete is its relatively high weight to load bearing capacity ratio. The UHPC concrete has been extensively used in a number of countries around the world. The first full size UHPC

Bridge was built in Canada in 1997. Several UHPC-bridges have been constructed on the European continent, such as France, Germany, Switzerland (rehabilitation & strengthening), Austria and Czech Republic. Worldwide countries such as the US, Canada, Australia, Japan, China, Malaysia also have UHPC bridges.

2.3 Requirements and standards

In Sweden, there are approximately 30 000 bridges with a span length exceeding 2 m, including culverts (*Mattsson 2008*). The Swedish Transport Administration (STA) is responsible for 20 900 of these bridges, of which about 16 500 are road bridges (*STA 2017a*). In total, 50 percent of these are built between 1950 and 1980 and have, thus, reached an age at which the need of maintenance and repair usually increases (*Silfwerbrand and Sundquist 1998*). A large percentage (86%) of these bridges is concrete bridges.

The STA uses an integrated system and during the last decade, has transferred resources from corrective to preventive bridge maintenance (*Silfwerbrand 2011*). Presently, 10 to 15 percent of the budget is devoted to preventive maintenance whereas corrective maintenance, repair, and reconstruction comprise the remaining 85 to 90 percent. Preventive maintenance aims at measures to maintain the function of the bridge structure. A lot of the planned maintenance is bought up during a 4 or 5 year span and in the contract several of the “smaller” maintenance interventions is described whereas larger repair projects are contracted separately.

Since 1st of July 2013 the STA is the solely responsible for the commission of transport infrastructure works in Sweden. Therefore the STA uses a variety of documents pertaining to the design, construction, inspection and maintenance and rehabilitation of concrete bridges. Some of these documents are requirements or recommendations or describe different procedures. Different aspects have different requirements and usually they overlap or are dependent on each other. The requirements on bridge maintenance specified by the STA are either technical or operation procedural (*STA 2017b*).

The STA document TDOK 2016:0204 “Krav Brobyggande” (*STA 20016a*) & TDOK 2016:0203 “Råd Brobyggande” (*STA 2016b*) lists and describes the requirements and advise for the design and dimensioning of new bridges constructed for the STA. Municipalities may also base their requirements on this document. For repair works they may have the same demands or based on the ones in the document. Some of them are:

- The use of sliding formwork is not allowed for cast in-place concrete.
- Concrete structures shall be dimensioned for a technical service life of 40, 80 or 120 years, according to the principles described in EN 1992-1-1 (*CEN 2004*).
- For each concrete surface, the exposure class that provides the most stringent requirement shall apply.
- In addition to the requirements in EN 1992-1-2 (*CEN 2004*), for road bridges, a minimum of 65 mm cover thickness applies in the cases where reinforcement corrosion may occur.
- In roads subject to the use of de-icing salts, bridge edge beams shall be treated with hydrophobic impregnations for prevention against water and chloride ingress.
- Concrete structures exposed to de-icing salts or marine environment shall be provided with the possibility electrochemical monitoring of the reinforcement.

The STA document TDOK 2013:0267 “Bärighetsberäkning av broar” (*STA 2017c*) specifies the requirements that apply to load calculations of bridges located on roads, railways and pedestrian and cycling routes. It defines, among others, 1) minimum and/or characteristic values for compressive strength, tensile strength and modulus of elasticity; 2) verification procedures (e.g. location of sampling, size and number of samples); 3) models for the calculation of deformations due to shrinkage or 4) maximum crack width for the different types of bridges.

AMA is a general material and work description in the construction industry. In AMA Anläggning 17 (*Svenskbyggtjänst 2017*) there are a number of repair procedures described for civil engineering structures. Chapters BED.141 and BED.14 stipulates demolitions by water jetting as required in TDOK 2013:0415 (*STA 2017b*). For each new edition of AMA the STA makes a new version of own additional requirements (e.g. *TDOK 2017:0441, STA 2017f*).

BaTMan stands for Bridge and Tunnel Management and is the database as well as management tool for the bridges in Sweden. Own and managed by the STA and many municipalities. For each structure there is information about type, ownership and design, including materials. There are also a number of documents concerning maintenance and management. The STA has a lot of rules and requirements on what needs to be documented in BaTMan before the repair can be performed.

The decision related to the type and frequency of inspection, maintenance, repair and rehabilitation operations is a complicated process; it is therefore important to identify the performance requirements that must be considered and fulfilled to ensure a quality repair. These include both technical (service life, durability, structural stability and safety, execution of work, etc.) and non-technical aspects (legal, economic, social, and environmental, etc.).

The service life and durability of the repair is expected to be at least the same as that of the original structure. This will depend on the exposure environment, the choice of repair materials/methods and on the quality of the execution of the repair works. Of great importance are the mechanical and transport properties of the repair material, the durability of the bond between repair and old concrete (very dependent on the accurate preparation and application) and the compatibility between old and new materials (e.g. differences in shrinkage properties between the materials can lead to cracking). Requirements associated to service life of the repair system are: carbonation and chloride diffusion rates; frost resistance; limited cracking; and bond/adhesion strength.

By restoring the loading capacity of the structural members, after repair, the structural performance is expected to be at least the same as the new structure. When restoring the initial strength, the loss of cross-section of the member by removal of deteriorated concrete must be considered and reflected on the design of the repair and choice of repair material (e.g. use of textile reinforcements or type of concrete) in order to accommodate the redistribution of the loads. Requirements associated with structural stability are: compressive and tensile strength; bond strength between repair material and reinforcement; E-modulus; amount and depth of surface scaling; and (residual) cross-section of the reinforcement.

Execution time depends on the extent of the repair and climatic conditions. It is important to minimize the execution time to reduce economical and societal impacts. Simultaneously enough time for careful application, dependent on the type of repair material/method, is required. This is of great importance in the case of temperature or moisture sensitive repair materials. Requirements related to execution have to do both with the correct execution itself but also with the adequate choice of methods. These include: methods for removing old concrete and their influence on the surface quality and bond to the repair material; methods and materials for injecting cracks and their influence on the permeability of the concrete; methods for placing the repair material and their influence on the bond to reinforcement and old concrete.

Environmental impact and health and safety aspects of the repair procedures should also be considered during its execution. Examples include energy consumption, waste management strategies, selection of materials with low environmental impact, personal protective equipment and protection of users against eventual debris. The environmental impact of the repair material must be considered with respect to its lifetime, in particular in terms of their effect on ecology as some repair materials can be highly toxic and their waste need special treatment.

2.4 Standards

There are a vast number of international and European standards pertaining to repair and maintenance of concrete structures, the most relevant of which are the series ISO 16311 parts 1-4, the series EN 1504 parts 1-10, and the EN 14487 parts 1-2.

The standard ISO 16311 provides the framework for maintenance activities of existing concrete structures, specifies the requirements for assessment of structures including inspection and evaluation of the performance, identifies key stages in the planning and designing of the repair process and specifies how the application of products and systems for concrete repair shall be conducted and verified.

The European standard EN 1504 specifies the requirements for the identification, performance and safety of the various techniques, systems and products used for the repair and rehabilitation of concrete structures. More detailed information is provided in section 5 of this report where an overview of the repair principles, techniques and materials for concrete structures is described.

The European standard EN 14487 pertains to sprayed concrete, a widely used material in reparation and maintenance works. Part 1 describes the requirements for sprayed concrete, constituent materials, composition and properties (fresh and hardened) whereas Part 2 pertains to execution and quality control.

With regards to UHPC, however, until very recently there were only technical guidelines and recommendations available in e.g. France (*SETRA-AFGC 2002*), Japan (*JSCE 2006*), USA, Australia (*Gowripalan and Gilbert 2000*) and Switzerland. Legally these documents were not official standards. Recently however, official national standards were published in France covering both materials as well as design (*AFNOR 2016a-b*). A third standard covering “Execution of concrete structures – specific rules for UHPC” is expected to be published at the end of 2017 (AFNOR 2017).

2.5 Repair techniques for concrete structures

Concrete structures are designed based on different requirements such as service life, safety, functionality, aesthetics etc. To maintain these requirements throughout the expected service life of structures a maintenance policy and inspections are needed.

If the inspection reveals that the structure does not fulfil the requirements, actions need to be taken. The EU project REHABCON (2004) distinguishes between four different types of actions, namely:

1. Do nothing at present, i.e. postpone repair for a certain time.
2. Issue restricted use.
3. Repair now.
4. Demolish and re-build.

The EN 1504-9 (*CEN 2008*) has the same overall principal but details the actions further in intermediate options. Weather to repair or replace a structure is an important issue that is addressed in an asset management system. There are several systems more or less similar. The following is a comparison between the repair and maintenance management systems described in the REHABCON project, the EN 1504-9 and the US Bureau of Reclamation (USBR).

The suitable repair option depends on the cause, type and location of the damage, so for each cause and type of damage, one or several repair techniques may be applicable. According to “Reparations Handbook” (*Betongreparation.se*) and “REHABCON Manual” (*REHABCON 2004*), there is 13 main repair techniques:

1. Concrete recasting
2. Local patch repair
3. Additional cement based cover
4. Surface treatment
5. Crack injection
6. New post-tensioning reinforcement
7. Strengthening with carbon fibre
8. Strengthening with steel plates
9. Cathodic protection
10. Corrosion inhibitors
11. Chloride extraction
12. Realkalization
13. Demolishing/replacement

Each technique is then divided in several actions.

	According to “Reparations Handbook” (*Betongreparation.se*) the following moments are fundamental part repair techniques 1-3:	**According to REHABCON Annex 4, execution of repair work is divided in a number of actions:**
1.	Removal of damaged concrete	Removal of damaged concrete
2.	Cleaning the concrete surface	Application of material replacing the old concrete
3.	Crack injection	Injection
4.	Cleaning of reinforcement, additional reinforcement and application of inhibitors	Replacement and additional reinforcement
5.	Application of stirrups	Use of chloride extraction and re-alkalisation
6.	Material selection	Application of cathodic protection
7.	Conditioning of the concrete’s surface	Cleaning of concrete surface
8.	Formwork application	Drying of concrete surface
9.	Concrete application (casting, spraying, etc.)	Application of non-structural surface layer on the cover
10.	Setting and hardening	Application of structural surface layer on the cover
11.	Surface treatment	Cleaning of reinforcement
12.	Consideration of traffic vibrations	Application of inhibitors
13.	Quality control	

State-of-the-art maintenance and repair solutions rely on the application of polymer-based cementitious materials, conventional reinforced concrete and hydrophobic surface treatments. These solutions bring a series of problems associated with compatibility issues between conventional concrete and polymer-based systems, poor durability associated with conventional reinforced concrete and environmental aspects related to the impregnations.

The ideal repair solution would be to use a Portland cement based material which would ensure excellent durability in harsh environments as well as superior bond to the existing concrete surface (*Brühwiler and Denarié 2013*). The solution should be also easily produced by any concrete plant and cast as self-compacting material using conventional methods known to contractors, while not requiring any additional heat treatment. Moreover, the strength development should be rapid to enable fast repairs. A promising material which could appropriately fulfill these above requirements is ultra-high performance concrete (UHPC) is an alternative solution which is rapidly spreading around the world to repair and strengthen concrete structures (*Denarié 2009, Cwirzen et al. 2008, Žnidarič 2003*).

Recent developments showed indeed that retrofitting and strengthening of concrete bridges by casting of UHPC toppings became a viable solution. For example in Switzerland between years 2004 - 2013 more than 25 applications were successfully completed. Over 100 m^3 or

3000 m^2 were laid. There was also one full scale application in Slovenia. This technology is rapidly spreading in Switzerland at the moment and becomes a “current practice” (*Brühwiler and Denarié 2013*).

UHPC is potentially an excellent material for applications in infrastructure (especially bridges) exposed to harsh environments.

Existing guidelines and recently published French standards for UHPC should enable their straightforward adaptation in Sweden after experimental and theoretical research performed in this project.

3 MATERIAL DEVELOPMENT

3.1 Material

UHPC require quality material to achieve the high strength. In traditional UHPC the cement quantity is high. One aim was therefore to reduce the cement content with supplementary materials. LTU developed recipes with quartz and lime while RISE/CBI used fly ash.

Materials used in the project include:

- Cement
- Silica fumes
- Fly ash
- Quartz filler
- Limestone
- Super plasticizers
- Water
- Aggregates < 4mm- different types
- (steel fibres)

3.2 Mixture development

3.2.1 Restrictions and criteria

To make the UHPC more environmental by reducing the cement content, a goal of 500-700 kg/m^3 cement content with a flow spread of ≥ 270mm (HägerManns cone) and a compressive strength of at least 120 MPa at 28 days. For concrete with supplementary materials the strength increase after 28 days may be higher than for traditional concretes with only cement.

After initial testing, it was decided both project partners would develop different mixtures using different supplementary materials. LTU would focus on creating a mix with limestone and CBI would optimize a mix with fly ash.

3.2.2 Mixdesign

The mix design started with testing the influence of some materials such as aggregates, supplementary material, water content, water content, curing & aggregates.

Initially different cements, supplementary materials, and aggregates were tested both in content and types to examine the effects on the flowability & compressive strength. Some of the results can be seen in figure 3-1.

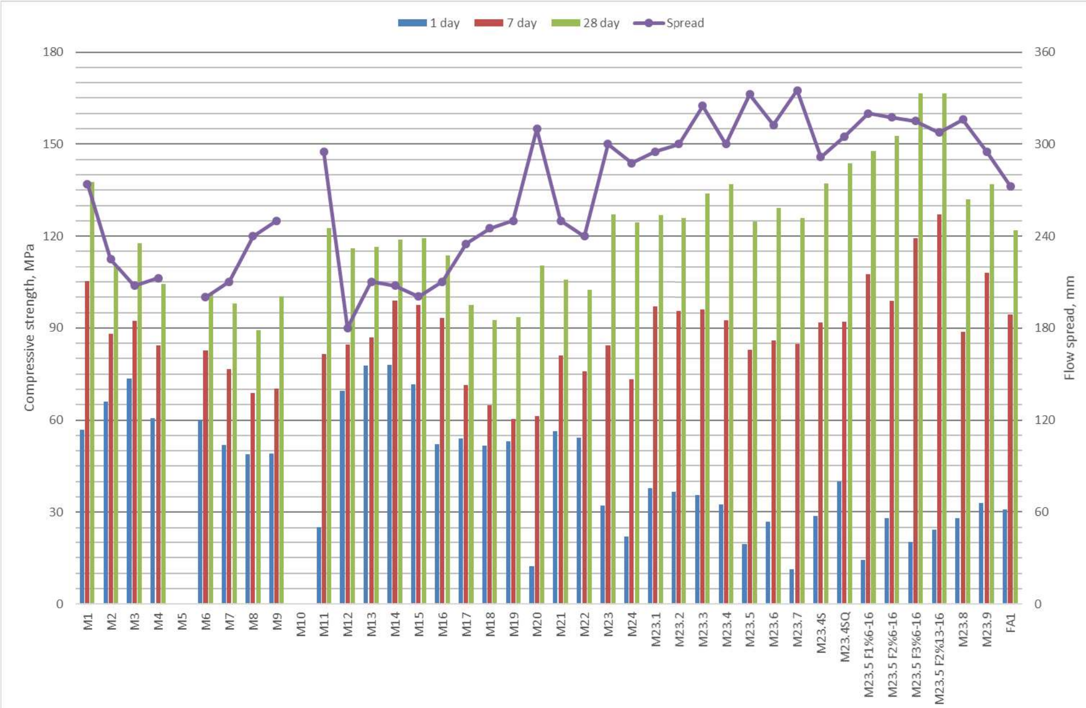

Figure 3-1 Result of spread flow and compressive strength for initial recipe development (From CBI)

After some initial testing the optimization for ecological UHCP with the cement reduction were carried out. The changes were smaller in this final stage and the focus were more on increasing the ecological effects while achieving the pre-determined conditions. Some of the initial results can be seen in figure 3-2. In the end the best ones were picked as the final mixes and tested for both durability and as a lay-over.

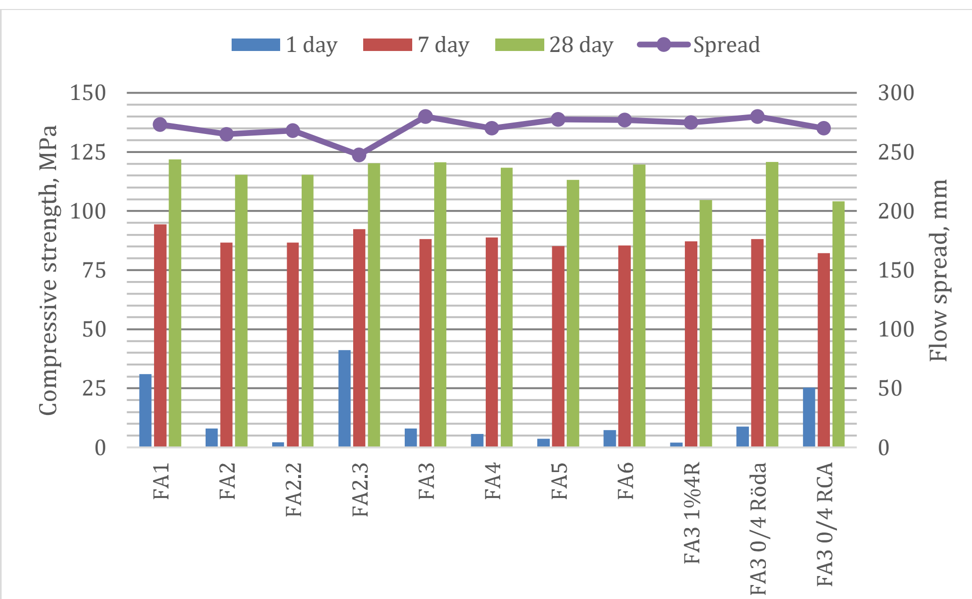

Figure 3-2 Results of flow spread and compressive strength for determine the final recipe from CBI

3.3 Final mixtures

3.3.1 Mix design

The final mix designs for CBI & LTU can be seen in Table 3-1. Both have a reduced cement content and supplementary material of fly ash or limestone and quarts filler.

Table 3-1 Final Recipes used for overlay testing

	Density [kg/m^3]	CBI FA [kg/m^3]	LTU 8 [kg/m^3]
ANL (CEM I 42,5 N -SR 3 MH/LA)	3200	550	664
Fly Ash (H10)	2550	400	
Silica Fume (920D)	2230		133
Silica Fume (971U)	2230	100	
Lime Stone (L25)	2700		664
Quartz Filler (M500)	2650		66
Steel Fibers (6/16)	7800	78	66
Steel Fibers (13/20)	7800	101	100
B20 (Aggregatets)	2650		232
B44 (Aggregatets)	2650		232
Röda 0/4 (Aggregatets)	2650	1150	
HD10 (Super plasticizer)	1090	25	34
Water	1000	140	184
w/c		0,282	0,308
w/b		0,148	0,257
w/s		0,065	0,095
Volume [m^3]		0,993	0,950

3.3.2 Compressive & flexural strength

The results from mechanical tests can be seen in Table 3-2. Testing have been done on samples with fibres, results for CBI FA without fibres are also presented as comparison. All meeting pre-required values.

Table 3-2 Compressive and Flexural strength of the final mixtures

Mixture	Flow spread [mm]	Compressive strength [MPa]		E-modulus [GPa]	Flexural strength [MPa]
CBI FA	280	7 days	88		
		28 days	121	47,3	5,9 (4-p bending)
CBI FA 2,5%		28 days	133	49,6	17,9
	Slump				
LTU 8_6F8	960 mm	1 day	67,4		
		7 days	101,1		
		28 days	130,4		17,8
LTU 9_6F3	770 mm	1 day	65,6		
		7 days	98,8		
		28 days	132,1		19,4

Figure 3-3 Samples after flexural testing, CBI FA 2,5% .

3.3.3 Shrinkage

The autogenous shrinkage is larger than the drying shrinkage for UHPC, almost twice as much, as seen in Figure 3-4 & 3-5. The reduction of shrinkage when adding aggregates and then additional fibres can be seen in Figure 3-6 & 3-7.

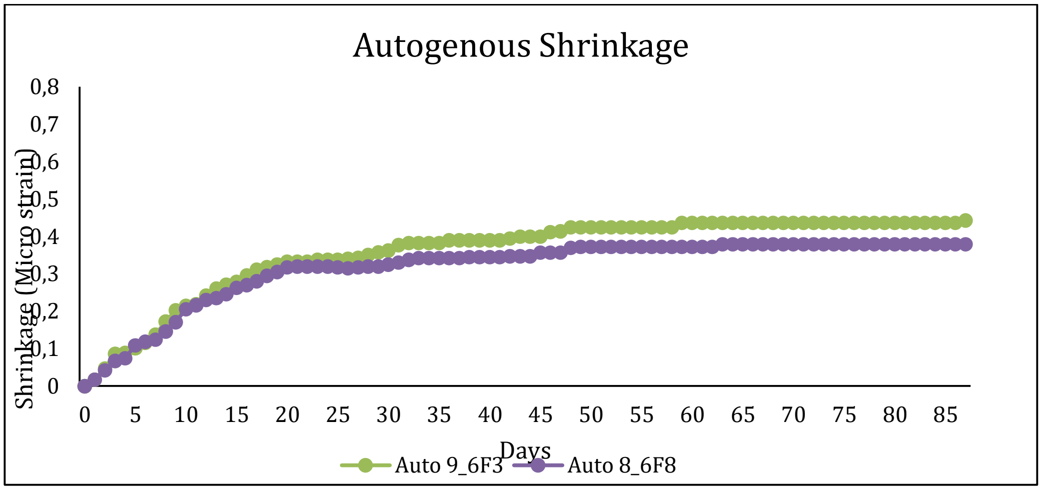

Figure 3-4 Autogenous Shrinkage of LTU final two recepies

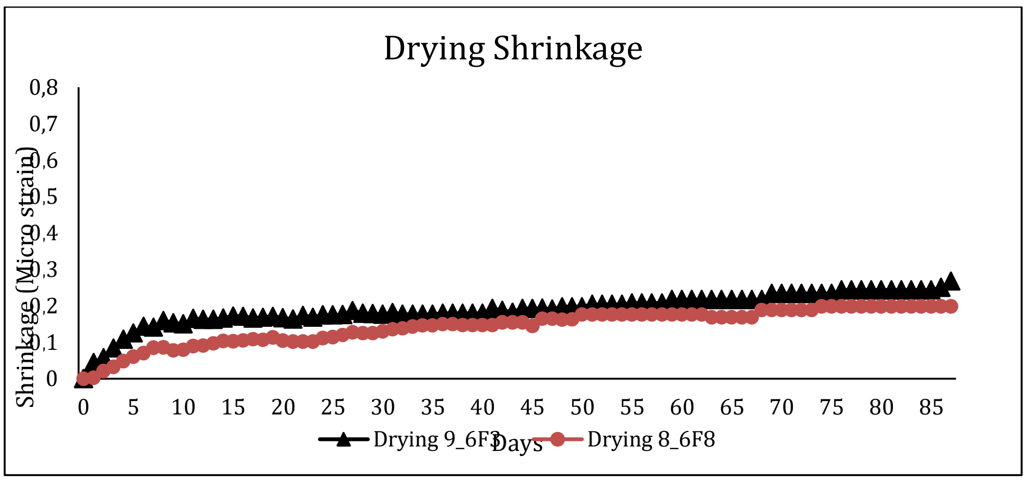

Figure 3-5 Drying shrinkage of LTU final two recipes (made at LTU)

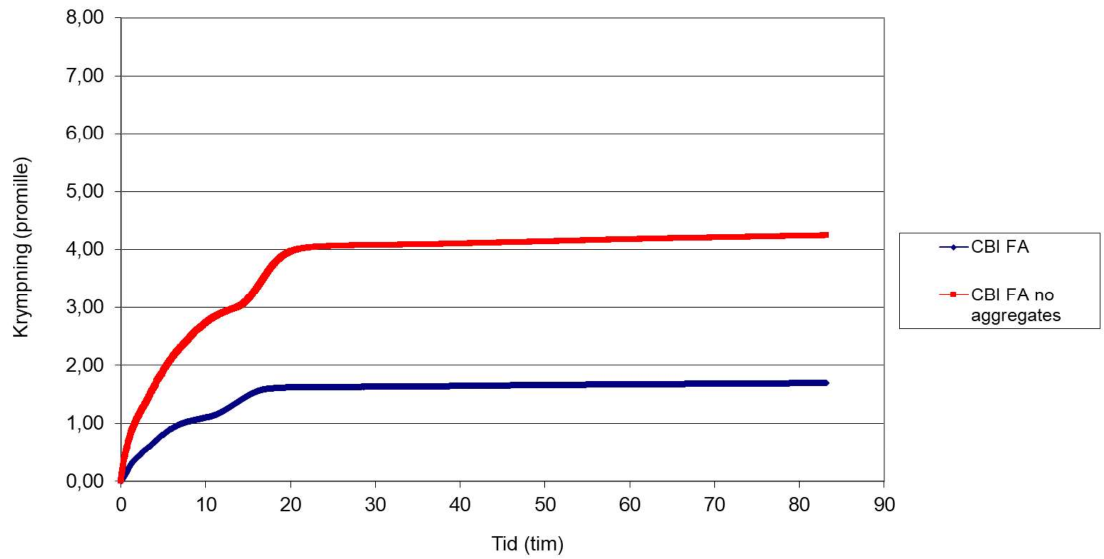

Figure 3-6 Autogenous shrinkage of LTU final two recipes (made at LTU)

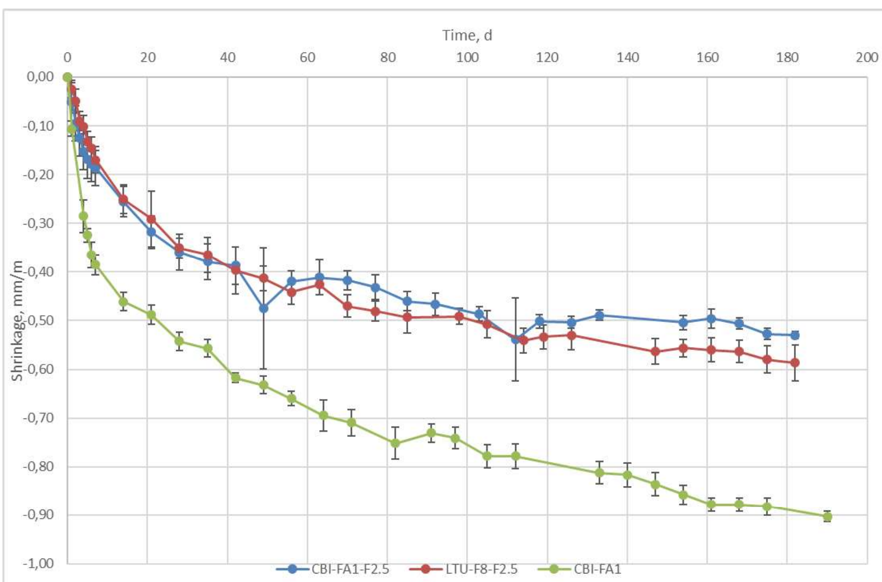

Figure 3-7 Drying shrinkage of recipes used for overlay testing as well as CBI recipe without steel fibers

3.3.4 *Durability*

The intended use of the UHPC is in a harsh environment and without any or almost no maintenance. The durability resistance required of the material is therefore high. The two recipes used for UHPC overlay were tested for chloride and freeze-thaw.

3.3.4.1 CHLORIDE RESISTANCE

The chloride migration coefficient were tested according to the standard NT BUILD 492. In Figure 3-8, The difference in chloride migration coefficient in comparison to a reference concrete of C37/40 is shown. UHPC have a much lower chloride migration coefficient and thereby much larger chloride resistance.

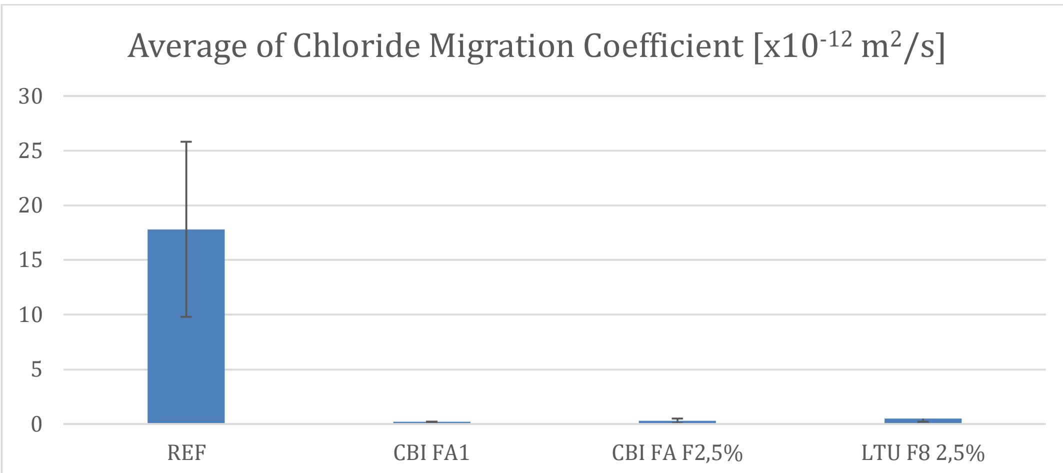

Figure 3-8 Chloride migration coefficient according to NT BUILD 419, with a reference as comparision.

In figure 3-9 the differences between the UHPC’s. The differences are small, and the standard deviation put the samples within each other. Any comparison between them is therefore not able.

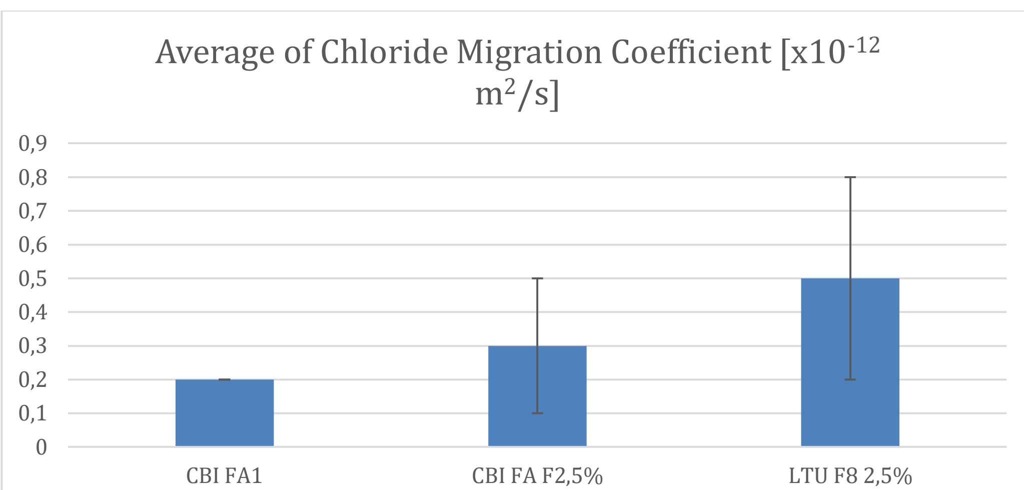

Figure 3-9 Chloride migration coefficient according to NT BUILD 419,comparision between UHPC.

3.3.4.2 FREEZE-THAW RESISTANCE

The freeze- thaw resistance was tested according to SS-EN 13 72 44.

The scaling for the samples with fibres were both weight as is and then possible loose fibre were removed, WF stand for with fibres and WOF stands for without fibres. All the UHPC had very good frost resistance as can be seen in figures 3-10 & 3-11 below.

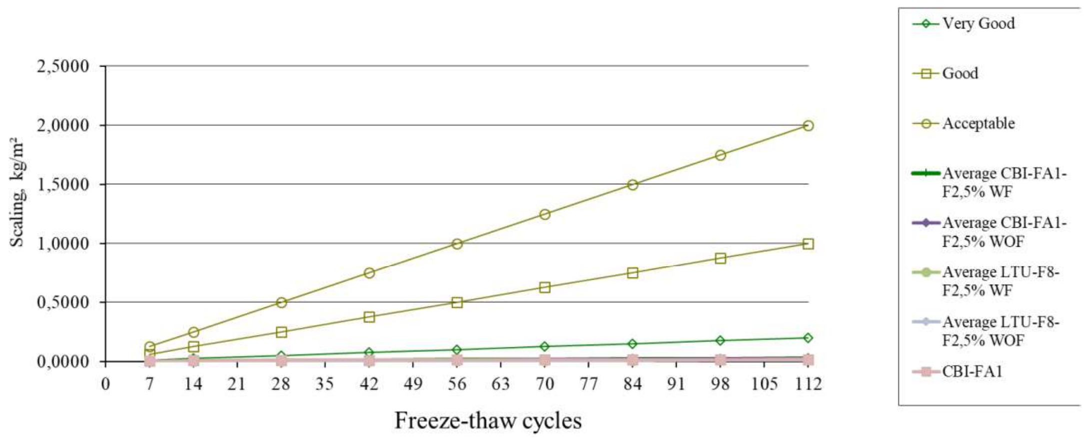

Figure 3-10 Freeze-thaw for final recipes, up to 112 cycles. All criteria shown

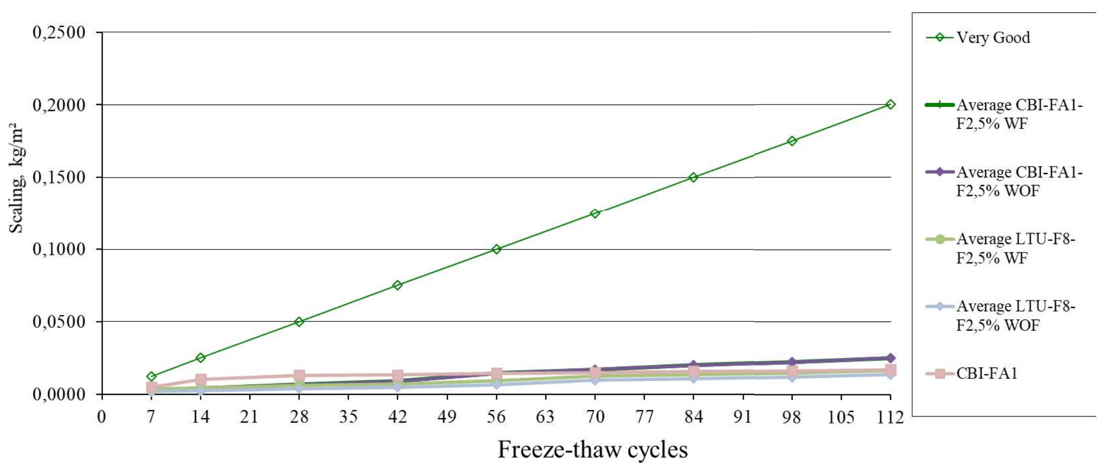

Figure 3-11 Freeze-thaw for final recipes, up to 112 cycles. Only best criteria shown

3.4 UHPC as an overlay

A successful reparation is not only dependent on the repair material but also on the bond and adhesion between old and new concrete. Two of the final UHPC recipes where therefore cast as a 20-30mm overlay on samples of reference concretes with different surface treatments. The primary reason for samples were to test the adhesion for different surface treatment with a fiber reinforced UHPC.

3.4.1 Reference concrete

The reference concrete had a water-cement ratio of 0,55 and a compressive strength of 40 MPa at 28 days. The concrete used CEM I (Anläggningscement) & crushed aggregates.

After more than three months of hardening the samples were split in two and treated with three different surface treatments:

- Grinding (a smooth surface)
- Milling (a rougher surface)
- Water milling (a rough surface)

As the surface treatment are important for the connection and strength in the transition zone, different types of roughness were examined. The surface treatments were performed by a contractor, which also resulted in a field test see 3.4.6.

Figure 3-12 Sample preparation at contractor. Grinding on the left and water jetting on the right.

3.4.2 Casting

The casting of UHPC overlay had special made casting molds, see Figures 3-13 & 3-14. The samples of reference concretes with different surface treatments were put in the molds and the UHPC with fibers were then cast. De-molded samples can be seen in Figure 3-15.

Figure 3-13 Form for overlay with different surface treatment of reference concrete. Tests: Shear, 4-point bending

Figure 3-14 Form for UHPC-overlay with different surface treatment of reference concrete. Tests: pull-out

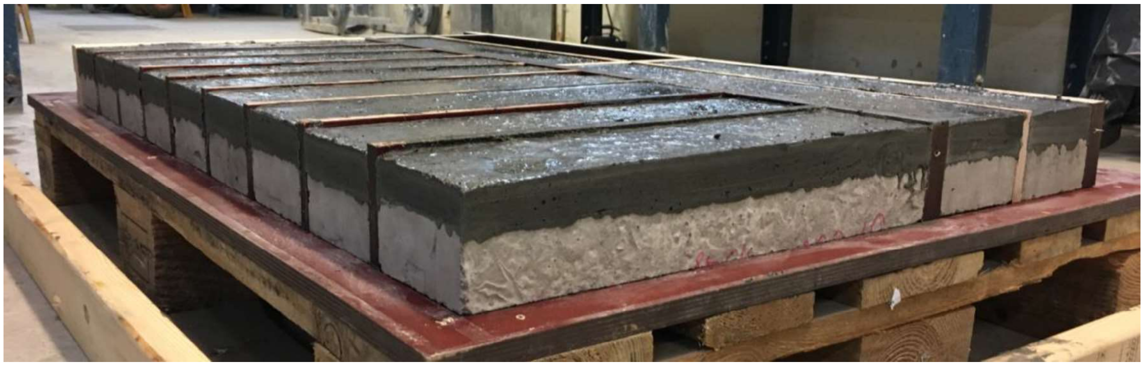

Figure 3-15 Demolded UHPC overlay specimens (prom picture 3-10)

3.4.3 Adhesions tests

Two types of adhesion test were performed to test the transition surface between the reference, old and weaker concrete, and the UHPC.

3.4.3.1 PULL-OUT TESTS

A common test for adhesion is the pull-out test. The procedure is as follows:

- A Ø50mm drills down to a certain depth. In this case the thickness of the overlay (25mm) and additional depth (40mm) to test directly at the zone, and in combination with the old concrete
- The metal pull pieces, see Figure-16 a & b, are fasten on the surface with a certain glue.
- The pull-out machine is then attached to the metal piece and will pull and measure the force and calculate the strength needed for failure. See Figure 3-17.

Figure 3-16 a) picture from above before pulling b) pulled specimen c) Pulling machine

There are two aspects that are important for adhesion, the strength and were the failure occurs. Figure 3-17 shows the pull-out strength of the two different mixes on the different surface treatment at two depths. Figure 3-18 shows the location of failures. Each is from 4 pull-out test except milling LTU 40 mm where one broke during handling & water grinding where one was unable to get a sample as the adhesion between glue and the metal block failed.

The result showed that the better (rougher) the surface treatment, the better adhesion. With increasing surface treatment, the better adhesion between old concrete and the new UHPC overlay.

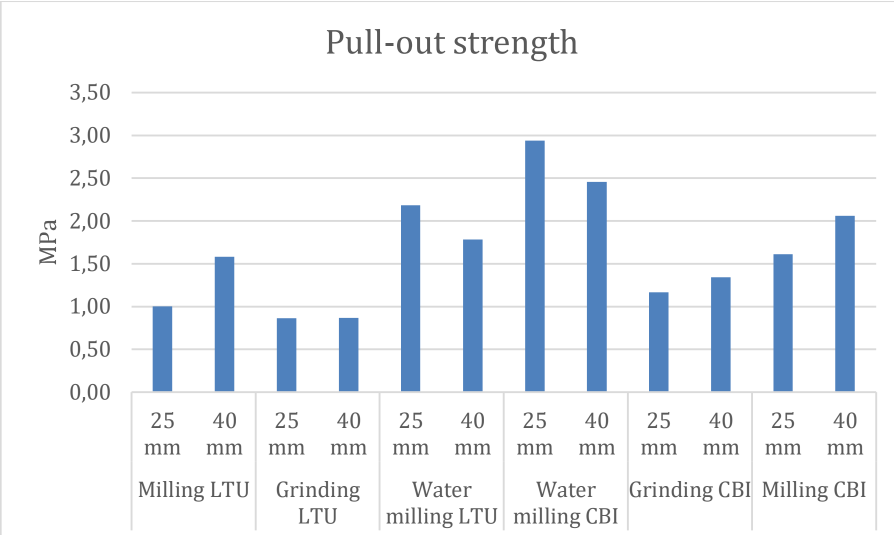

Figure 3-17 Strength at failure of pull-out tests. Medium of four specimens

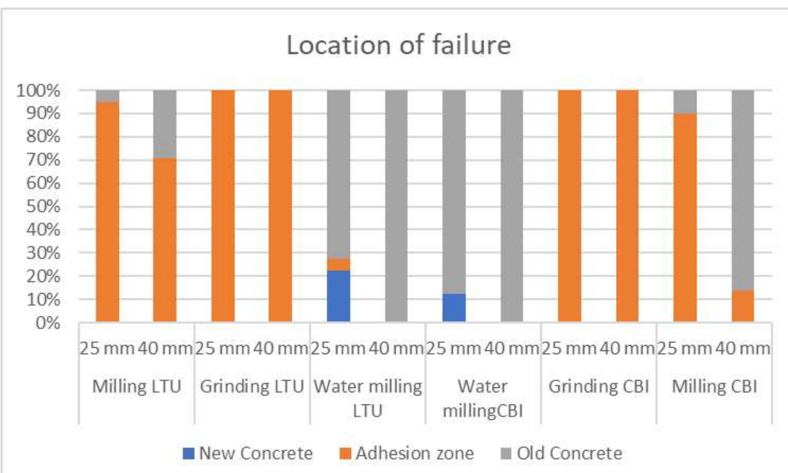

Figure 3-18 Type of failure for the pull-out tests. Based on four specimens

3.4.3.2 SHEAR TESTS

The shear strength of the bond were tested as seen in Figure 3-19. The test was set up to create a shear force in the adhesions zone between the reference concrete and the UHPC overlay, see Figure 3-19. Similarly, as for the pull-out test, both the failure type and strength is of interest.

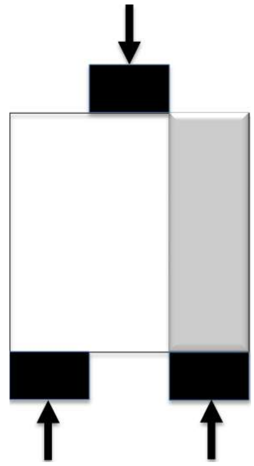

Figure 3-19 Shear test set up. The gray part on the right is the UHPC overlay.

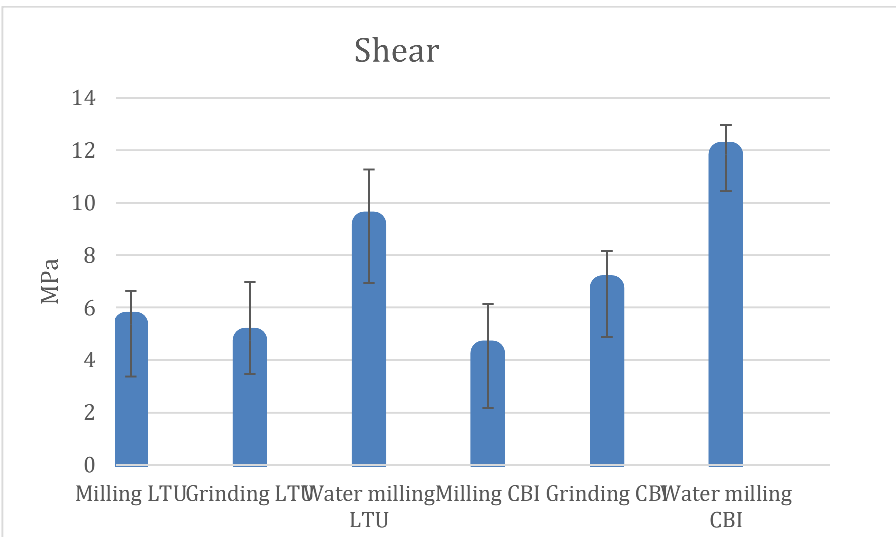

Figure 3-20 Shear strength of tested specimens

Not surprisingly, the better surface treatment the better shear resistance in both strength and in type and location of failure. As can be seen in Figures 3-20 & 3-21

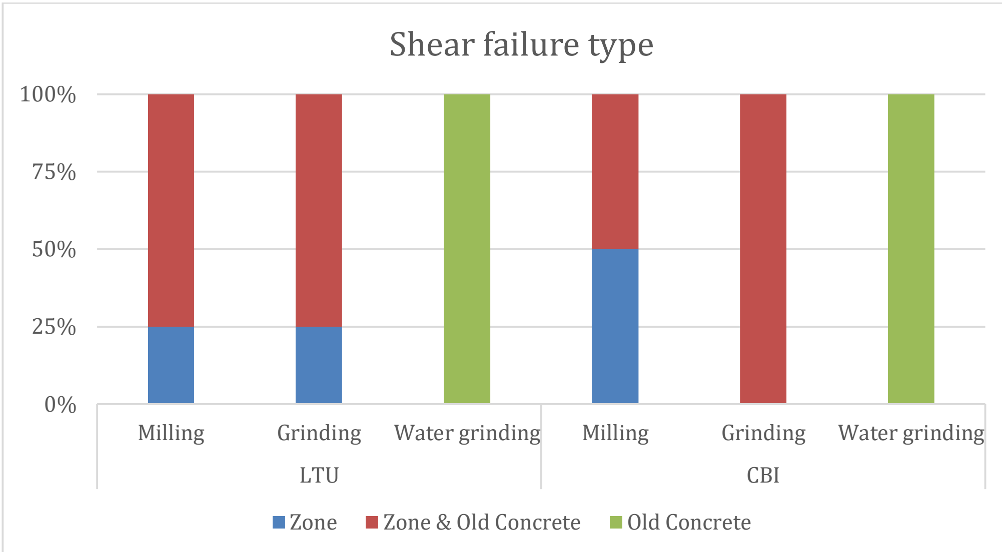

Figure 3-21 Failure zone of shear tests

3.4.4 Bending

A flexural test with UHPC as an overlay on the bottom half during tested showed no major differences in concern to flexural strength, probably as the majority of the flexural tension is taken up by the fibres in the overlay in the tension zone. The tests were conducted at a seven weeks of age partly to ensure maturity and of practical reasons.

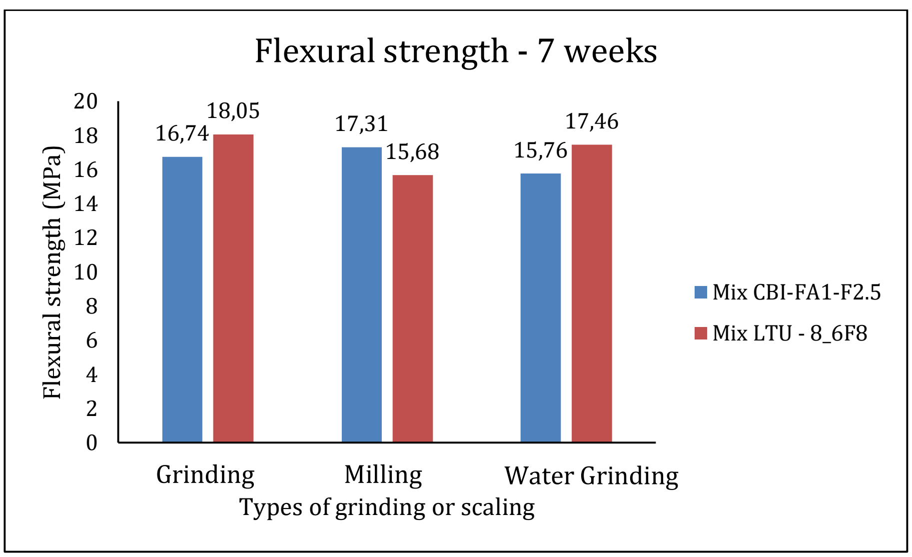

Figure 3-22 Flexural test of UHPC overlay at 7 weeks of age.

3.4.5 Field test

During the surface treatment of the samples used for UHPC overlay, the contractor offered the opportunity to do a small field test as the whole floor would be demolished on a site they were working on. The test surfaces were four 1x1m squares with the three different types of surface treatments and an additional fourth that had been water jetted deeper and with more exposed aggregates. Only three of the four squares were cast as one were deemed to shallow. The field tests main focus was to prove that mixing in field with a tumble-mixer was possible. All dry materials were transported to the site. Water were provided on site. No fibres were used.

Figure 3-23 Left: two water jetted surfaces, different depth. Right: The three (out of four) surfaces filled with UHPC

The mixing went well with the tumble mixer. The UHPC could then be pured on the sqares and filled out nicely. Just a finishing stroke at the end was enough. The mix worked as expected. Crushed aggregates were used and a slightly higher water/cement ratio was used in one of the test sqares. Additional water content from the sqares themselves increased the total water content.

4 LARGE SCALE TESTING

Large scale testing was performed at LTU with casting of fiber reinforced UHPC, mechanical & adhesion testing and FEM modeling.

4.1 Test setup

Large scale testing were performed on a 300x300x2500 mm beam/pillar. The specimens were cast as beams, water jetted and the repaired as a column, se Figures 4-1 to 4-7. The LTU 8_6F8 mixture were modified with an increased water-cement ratio to 0,33 and the cement content to 651 kg/m^3. The UHPC overlay were about 30 mm thick, variated due to uneven surface from water jetting.

Figure 4-1 Form work (300x30x2500mm)

Figure 4-2 Conventional concrete

Figure 4-3 Scalled (water jetted) cube

Figure 4-4 Scalled (water jetted) column

Figure 4-5 Form work, Overlay

Figure 4-6 Demolding

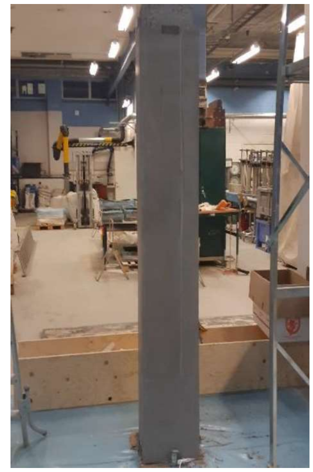

Figure 4-7 Finished column with UHPC overlay

4.2 Mechanical properties

The compressive strength of the normal strength concrete (NSC) and composite concrete were determined accordance to Swedish standard SS-EN 12390-3:2019 at the age of 7 and 28 days, accounting the day from UHPC-layer was casted. Similarly, the flexural strength (three-point bending) of specimen of dimension 360*360*2500 mm was determined after 28 days as per standard SS-EN 12390-5:2009. Figure 4-8 and 4-9 shows the compressive and flexural strength development of NSC and composite concrete.

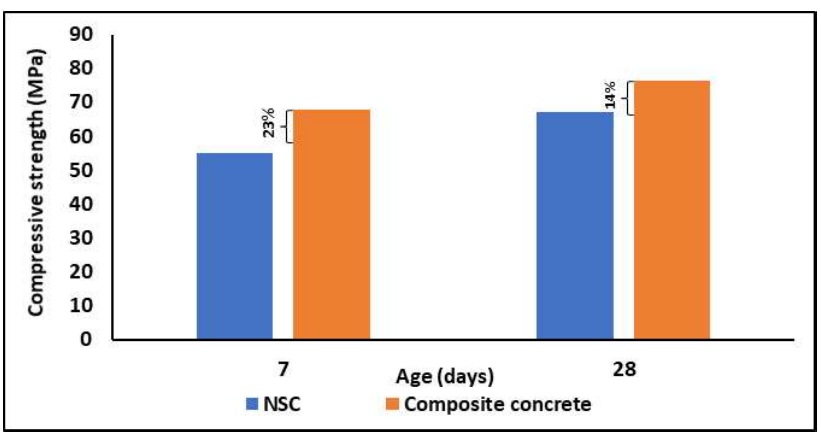

Figure 4-8: Compressive strength development of NSC and composite concrete

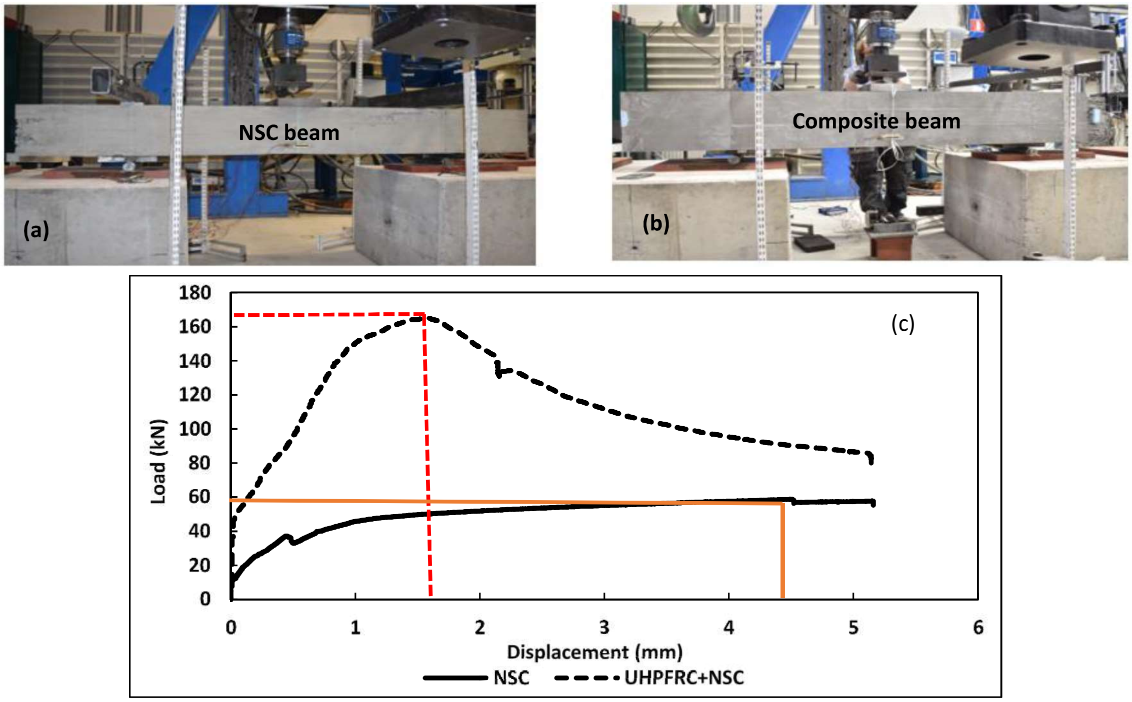

Figure 4-9: Three-point bending test setup (a) NSC, (b) composite concrete; (c) load vs displacement curve

Measured compressive strength of the composite concrete increased by 23% and 14% after 7 and 28 days respectively compared to NSC. This pronounced strength is evidently supported by a layer (15 mm) of UHPC around the treated concrete surface. Consequently, formed dense microstructure by filling the voids and cracks in NSC surface. Correspondingly, with an overlay of 30 mm UHPC, composite beam reached flexural strength of 8 MPa at 28 days. While the normal concrete specimen reached only 4.8 MPa. Most importantly no delamination was observed along the direction of loading, despite parallel to the bond surface. This is mainly inferred to the rough NSC surface, supporting the high friction and aggregate interlocking mechanism between the overlay and NSC concrete.

4.3 Bond strength

Interfacial bond strength between old substrate and repair concrete (UHPC) is examined using pull-off strength test as per standard ASTM C1583. Test is performed at four different locations on the beam i.e., extreme bottom (-90cm), bottom-center (-30cm), top-center (+30cm), and extreme top (+90cm).

Results shown that for every analyzed point, failure mode in NCS was observed (Figure 4-10) and is in line with the calculated bond strength values (> 2.1 MPa) which lies under the category – “excellent bond quality” as per standard ACI 546 (Figure 4-11). However, with increasing UHPC thickness a slight decrease in strength was observed. This is mainly attributed to the smooth coarse aggregate which was under the test zone (Figure 4-10 (c)) which did not interlock with the repair concrete and small variation in core drill depth also influenced early failure load.

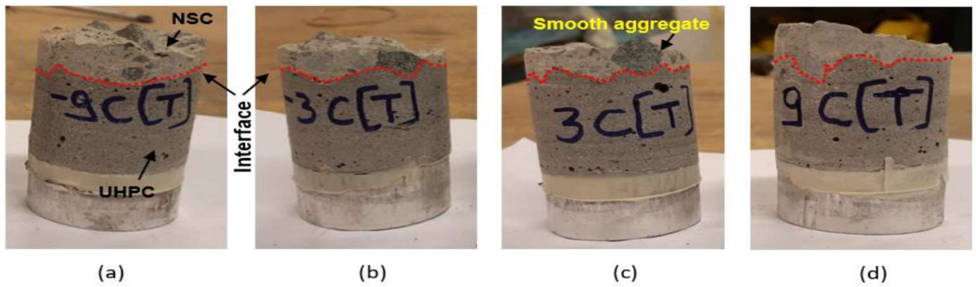

Figure 4-10: Bond failure and failure mode at – (a) extreme bottom; (b) center-bottom; (c) top-center; (d) extreme top

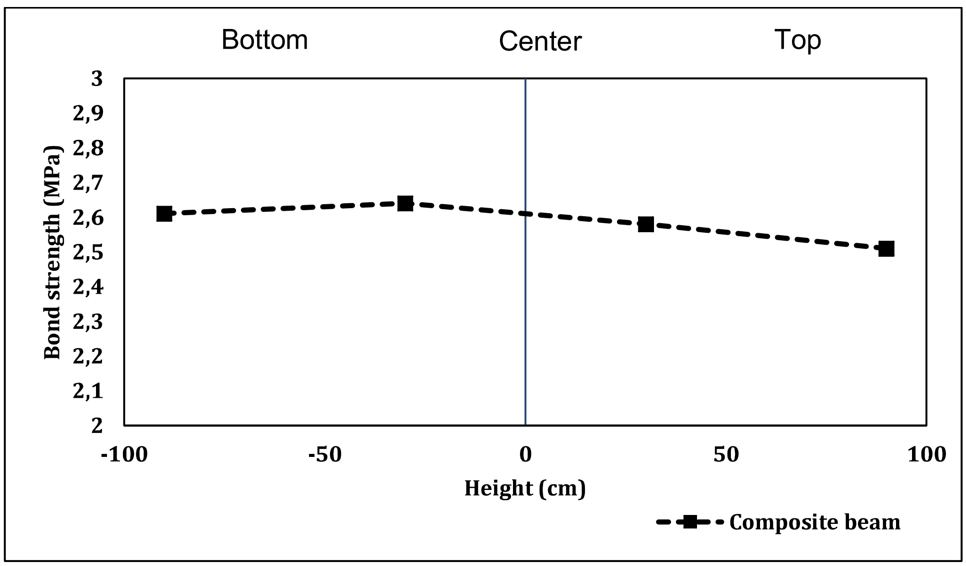

Figure 4-11: Bond strength - composite beam

4.4 Crack depth

Further shrinkage crack depth was assessed to see if the cracks has penetrated through the overlay UHPC. It was done by applying a layer of epoxy resin on the top surface of the cracks. Followed by drilling core of 35 mm diameter perpendicular to the surface, partially passing through NSC concrete. Using digital optical microscope, depth of cracks was visualized and measured. Figure 4-12 shows that the cracks did not penetrated through the NSC layer and in fact the development of cracks stopped almost 1 cm before the interface.

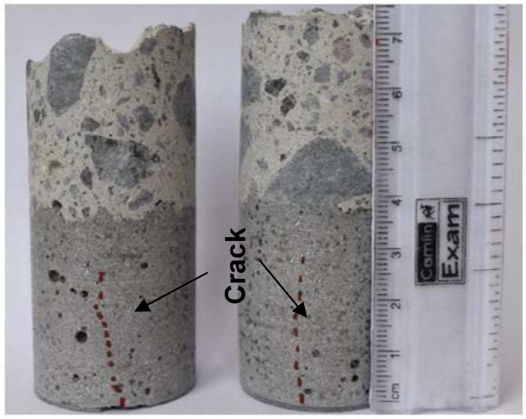

Figure 4-12: Cracks depth of composite concrete

4.5 Preliminary finite element model

The study was carried out on NSC and composite concrete beam specimens aiming to make a comparison between the laboratory results and the prediction from modelling by using the finite element method. The test beams are simulated on ATENA 3D v5 software. Program ATENA was selected because this program is specially designed for simulates the real behaviour of reinforced concrete structures. The parameters of the material used and details of specimens are shown in Table 4-1 and Figure 4-13. A typical figure of the three-dimensional finite element mesh of the studied beams is shown in Figure 4-14.

Table 4-1. Finite element model parameters.

Item		
Compressive strength of NSC, MPa		53.17
Compressive strength of UHPC, MPa		130.40
Avg. density of the NSC, kg/m^3		2449
Avg. density of UHPC, kg/m^3		2537
Ultimate tensile strength of reinforcing bar, MPa		400
Beam dimensions, mm	NSC	300x300x2500
	Composite (NSC+UHPC)	360x360x2500

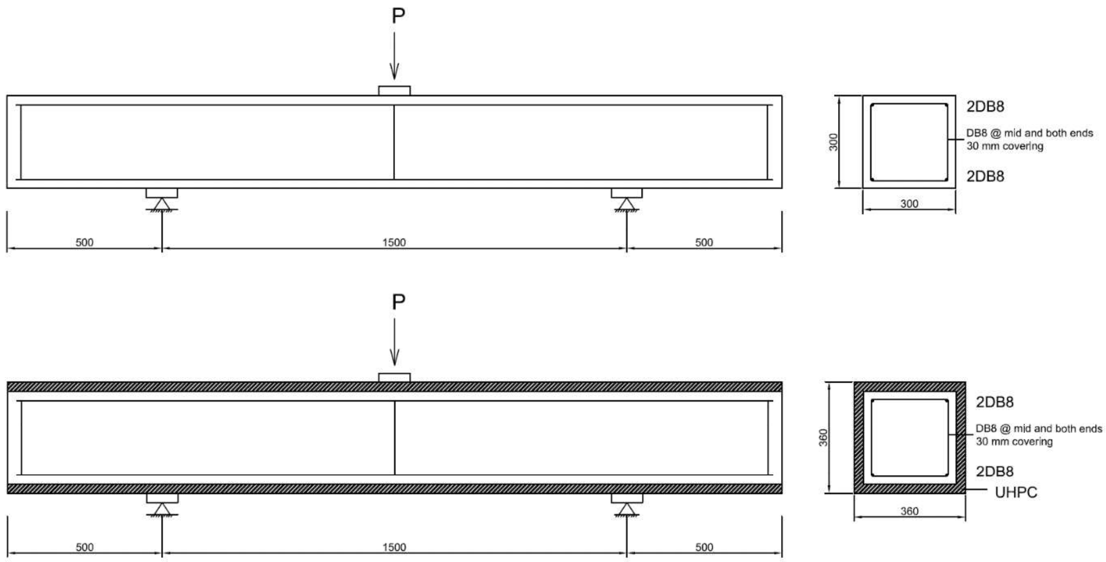

Figure 4-13: Experimental model details of NSC (top) and composite concrete (bottom).

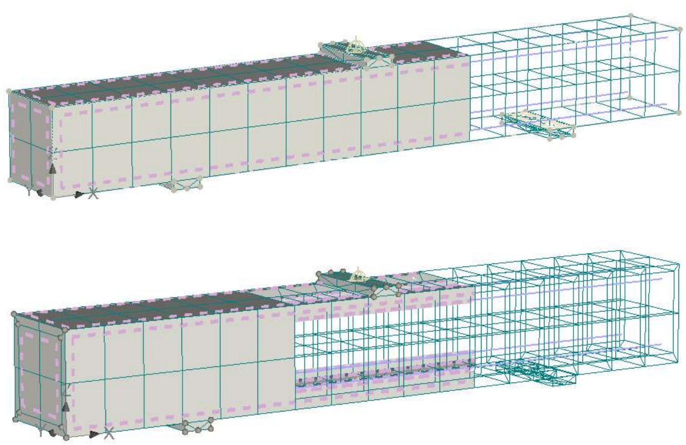

Figure 4-14: Finite element model of NSC (top) and composite concrete (bottom).

The analytical outcome of NSC beams and composite concrete beams are shown in Figure 4-15 and Figure 4-16 Both Figures showed a comparison between full-scale testing in the laboratory and FEM.

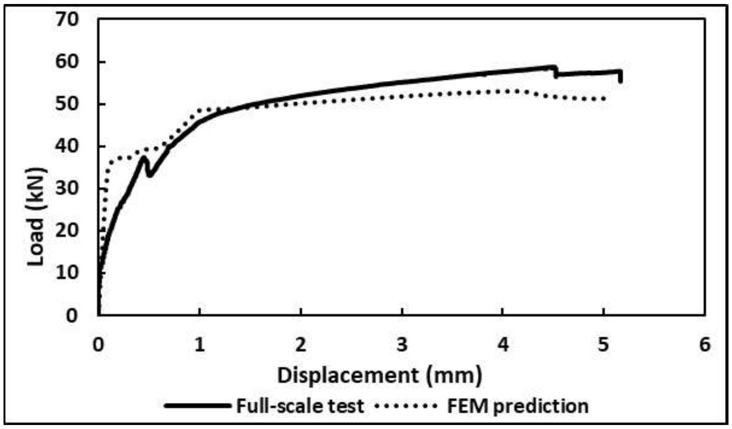

Figure 4-151: Load-displacement curve of NSC beam.

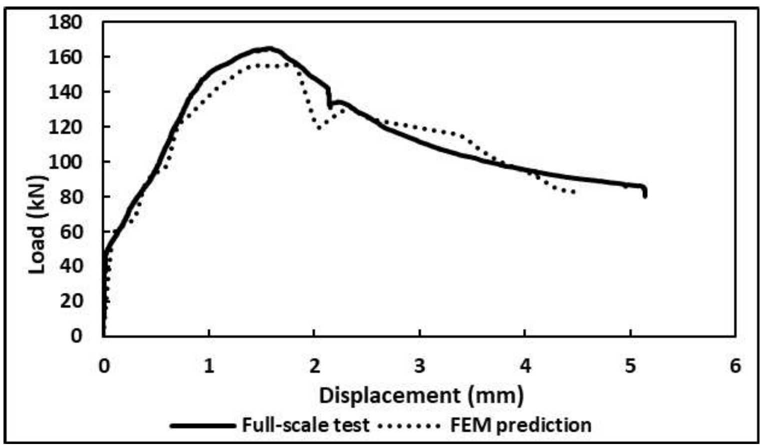

Figure 4-162: Load-displacement curve of composite beam.

The results from the finite element model of NSC and composite beams showed the ultimate failure load of 54.12 kN and 155.8 kN, respectively. The calculated failure load from FEM analysis result showed relatively 6% less value compared to full-scale testing. In addition, the macro-crack formation of full-scaled test beams and FEM analysis showed approximately 4-5 mm of crack thickness at the middle of both testing beams, see Figure 4-17 and Figure 4-18.

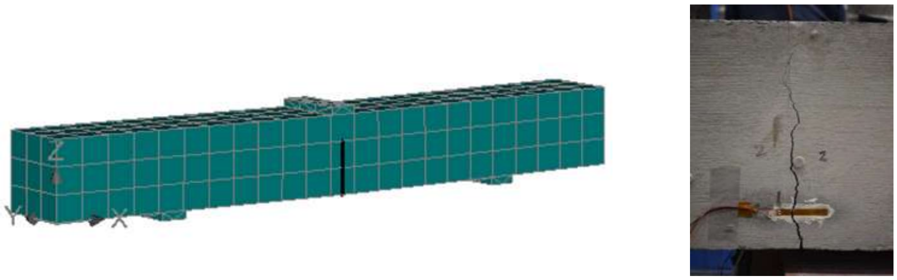

Figure 4-173: Macro-cracking of NSC from FEM (left) and full-scale test (right).

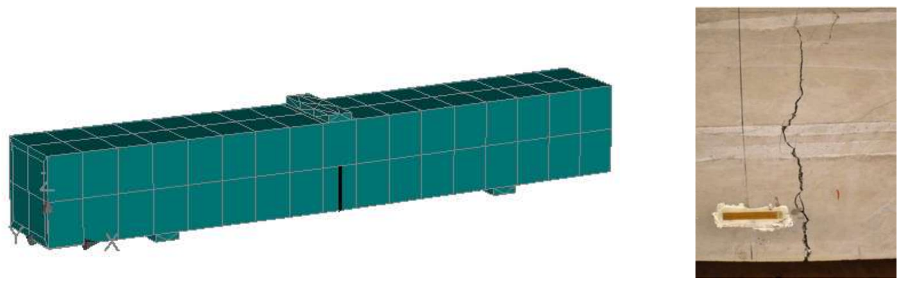

Figure 4-18: Macro-cracking of composite concrete from FEM (left) and full-scale test (right).

The finite element predictions of both the mechanical response (load and displacement at mid-span of testing beams) and macro-crack formation have been found to be in close agreement with existing test results.

5 LIFE CYCLE ASSESMEN/LIFE CYCLE COSTS

A simplified LCC and LCA study on CBI FA and LTU F8 have been conducted in comparison to a UHPC reference and a traditional repair concrete.

There are different ways to reduce the climate impact of concrete structures. One way is to use the resources more efficiently by reducing the total clinker content in the concrete mix. This can be achieved for example by replacing part of the Portland cement (CEM I) with supplementary cementitious materials (SCM) from industrial waste products such as fly ash (FA) and ground-granulated blast furnace slag (GGBS). Another way to reduce the climate impact is by increasing the durability of the structure and thereby extending the service life and reducing the need for repair and replacement. Müller et al. (2014) propose that the sustainability potential of a concrete structure should be defined as the relationship between the lifetime performance and the environmental impact. According to this definition the authors suggest three approaches to enhance the sustainability: 1) lowering the environmental impact of the concrete mix; 2) improving the concrete performance, i.e. reduction of cross-section of members through high load bearing capacity and 3) by extending the life span of the material and the structure. By reducing the need for repair and replacement there can be a significant reduction in environmental impact over the life span of a construction.

The concrete mixes chosen for this study are described in Table 5-1. The concrete mix for the traditional concrete is based on an EPD of Betongindustri's infrastructure concrete called FrostBI (EPD no. NEPD-1711-695-SE). There might be a slight difference between a repair concrete and a concrete for new construction as the former might contain more polymers and thus be more expensive and less environmentally friendly. By choosing a concrete for new structures the comparative cost and environmental impacts for UHPC are not underestimated

Table 5-1 Mix proportions of chosen concrete types.

Mix design (kg/m3)	UHPC ref	UHPC 1 (CBI FA)	UHPC 2 (LTU F8)	Traditional concrete (C34/45)
CEM I (Anläggning)	960	550	664	425
FA		400		
SF	190	100	133	
LL			664	
QF	285		66	
Steel fibers		179	166	
Aggregates	670	1150	465	1710
Superplasticizer	30	25	34	
Water	210	140	184	170
Properties				
w/c (-)	0,238	0,282	0,308	0,400
w/b (-)	0,198	0,146	0,257	0,400

Density (kg/m^3)	2345	2544	2377	2369
Compressive strength (MPa)	138	135	130	45

The structure which the cover thickness is applied on is a bridge pier with a diameter of 1 m and a length of x m. The pier is supporting a bridge over a highway and is therefore subjected to frost and deicing salt which comes from the moving cars. The corresponding exposure class is therefore XD3/XF4 and the exposure class is chosen accordingly.

The cover thickness used in this study are described in Table 5-2. The tolerances have been included in a sensitivity analysis.

Table 5-2 Cover thickness for a design service life of 100 years (corresponding 120 years according to STA).

	UHPC	**Traditional concrete**
Cover thickness (mm)	20	50
Cover thickness with tolerance (mm)	30	70

The chosen mix designs in Table 5-1 are evaluated by comparing the GWP and costs based on the following:

- Production of one cubic meter of fresh concrete.
- Replacement of cover thickness based on regulations and recommendations in order to achieve the same design service life.

This will be done through a life cycle assessment (LCA) and life cycle cost (LCC) assessment as described in the next sections.

Table 5-3 Lift-cycle stages divided into modules according to EN 15804

LIFE-CYCLE INFORMATION															
Production stage			**Construction process stage**		**Use stage**							**End-of-life stage**			
Raw materials	**Transport**	**Manufacturing**	**Transport to construction site**	**Construction**	**Use**	**Maintenance**	**Repair**	**Replacement**	**Refurbishment**	**Operational energy**	**Operational water use**	**Demolition**	**Transport**	**Waste processing**	**Disposal**
A1	**A2**	**A3**	**A4**	**A5**	**B1**	**B2**	**B3**	**B4**	**B5**	**B6**	**B7**	**C1**	**C2**	**C3**	**C4**

5.1 LCA

LCA is a systematic approach to assess the inputs and outputs of a product system throughout the whole life cycle. LCA can be prepared in several methods. The one chosen in this project is an attributional LCA focusing on global warming potential, which is expressed as kg CO_2-eq. The method follows ISO 14040 and EN 15804, but with one chosen indicator.

5.1.1 Background data

Two units are used for comparison between the concrete alternatives:

- Production of one cubic meter of fresh concrete.
- Replacement of cover thickness based on regulations and recommendations in order to achieve the same design service life.

The by-products fly ash and silica fume have a revenue which is less than 1 % of the total revenue compared to the main products as of first half year of 2020. This means that no environmental burdens are allocated to fly ash and silica fume in accordance to EN 15804.

It is assumed that the reinforcement does not need to be replaced during a concrete repair. It will only need to be cleaned. The concrete cover is repaired by removal and adding a new cover. It can be assumed that the work and preparations which is needed to repair the cover is the same for both UHPC and traditional concrete. In that case it will not be necessary for comparison.

Due to lack of information, it is assumed that the energy use at the concrete factory is the same for both UHPC and traditional normal concrete. Aggregates for traditional concrete have a short transport distance and is in this study set at 40 km. For the UHPC special types of aggregates are used which have a longer transport distance. All other transport distances are based on actual plants from the used products.The energy use for concrete repair processes has been chosen from Årskog, Fossdal and Gjørv (2004).

5.1.2 Impact assessment

When considering only the raw material production and transport of raw materials to concrete plants the climate impact is, as expected, higher for the UHPC concrete due to the larger amount of cement clinker used. Since the cement replacing materials have a low embodied climate impact the reduction will be noticeable. Figure 5-1 shows the GWP in kg CO_2-eq per cubic meter of fresh concrete. It can be noted that the UHPC 1 and UHPC 2 have a significantly lower impact than the reference UHPC, for the same performance. However, steel fibers, which are used for reducing shrinkage, have a remarkable contribution and without those the GWP could be reduced by 40 % of the UHPC reference, coming closer to the impact of the reference concrete.

Figure 5-2 shows the climate impact when considering also the energy use at concrete plant and transport to construction site. It can be noted that the raw materials have the highest contribution, although transport to factory cannot be neglected. The other life cycle stages, factory activities and transport to building site, have a much lower contribution in comparison.

Regarding the transport distances for UHPC, the biggest influences to the GWP are the distances of the special aggregates, fly ash, silica fume and steel fibers. The contribution of the transport of fly ash is notable in UHPC 1.

The previous LCA of the production stage highlighted the significance of the steel fibers. An additional analysis was therefore preformed in order to investigate the potential of the concrete types without fibers where the shrinkage is dealt with through an alternative approach. Figure 5-2 a) shows the material related climate impact of the concrete types without fibers while Figure 5-2 b) shows the climate impact during production and transport to factory. It shows that the UHPC 1 has a 50 % higher GWP than C 35/45 compared to the previous 94 % when A1-A4 is considered.

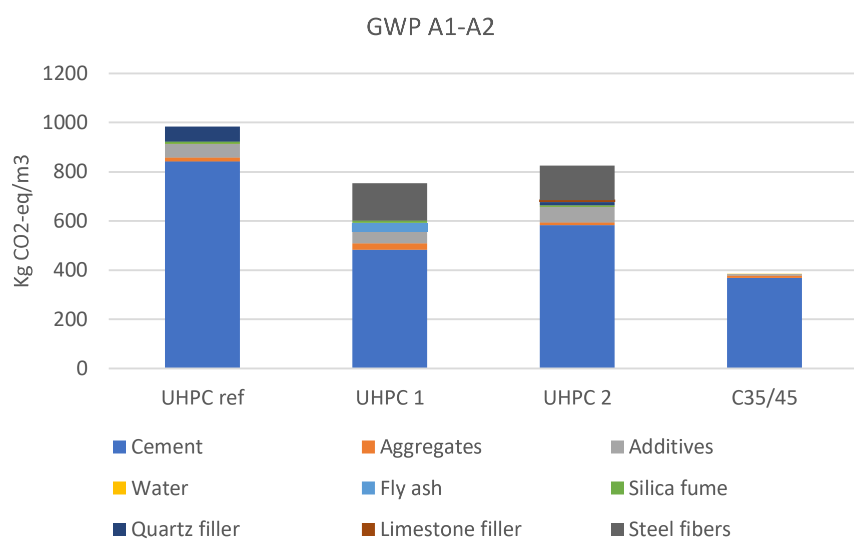

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Figure 5-1 Climate impact on the concrete mixtures during raw material production and transport to concrete plant

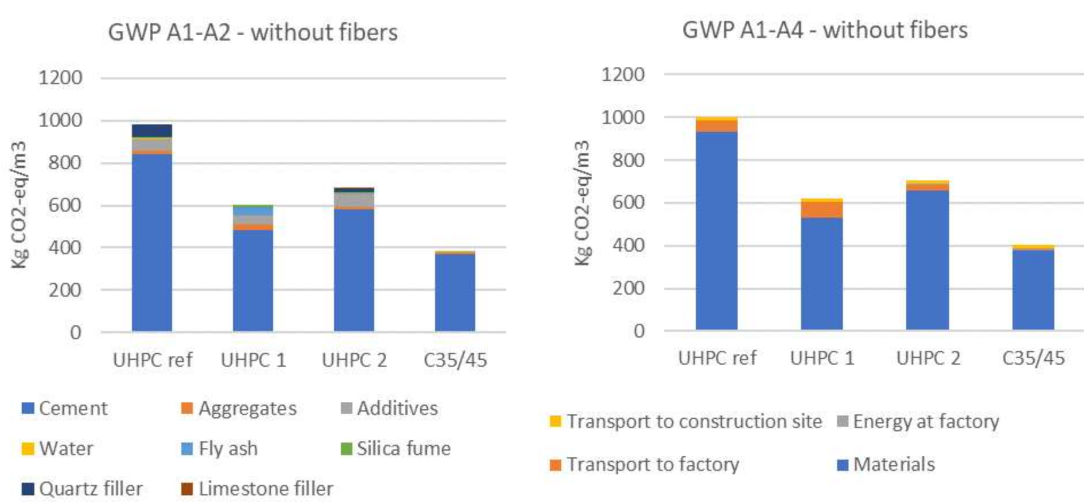

Figure 5-2 Climate impact of concrete mixtures without fibers during a)A1-A2 raw materials and transport and b) A1-A4 production and transport to construction site

5.1.3 GWP of production and transport of 1 m^2 of concrete

The first step of the analysis was to investigate whether the concrete mix of UHPC was climate optimised and how it could be further optimised. In this step the application of the concrete is considered in order to understand if the amounts used are enough to compensate for the higher climate impact of 1 m^3 of concrete. In Table 5-2 the cover thicknesses for XD3/XF4 in order to get a service life of 100 years was shown. When applying those cover thicknesses in the analysis it can be noted that the UHPC alternatives have a lower impact than the traditional C 35/45 concrete (Figure 5-4). Especially the UHPC 1 which has a 32 % lower impact.

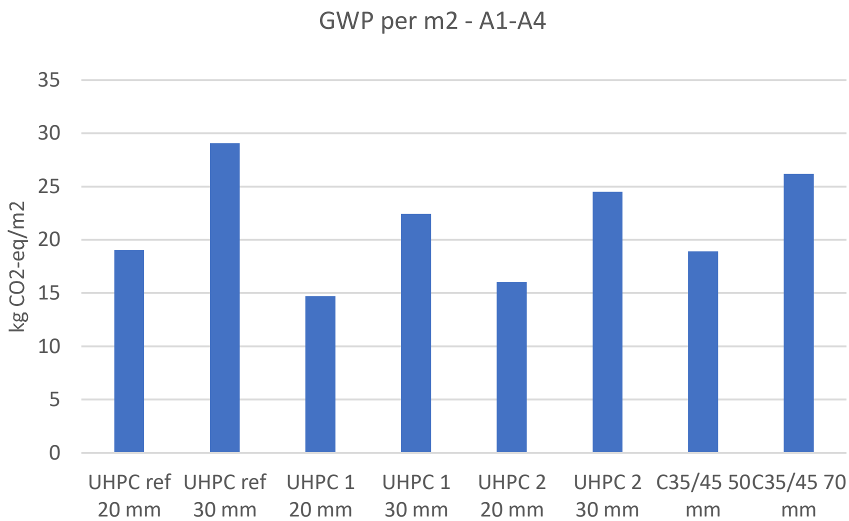

Figure 5-3 Climate impact of concrete cover with the chosen mixes during production and transport to construction site. With fibers.

5.1.4 GWP of repair

When considering the repair, it can be noticed that the material itself does not have the highest climate impact (Figure 5-5). Most of the energy consumption is due to hydro jetting. In this case, based on the study by Årskog, Fossdal and Gjorv (2004), the reduction in climate impact for avoiding a repair by using UHPC has a greater benefit than the direct emissions from the concrete itself. Therefore, the durability of the concrete cannot be neglected in an LCA analysis. The figures are valid for an average repair depth of 50 mm which is in line with the current study.

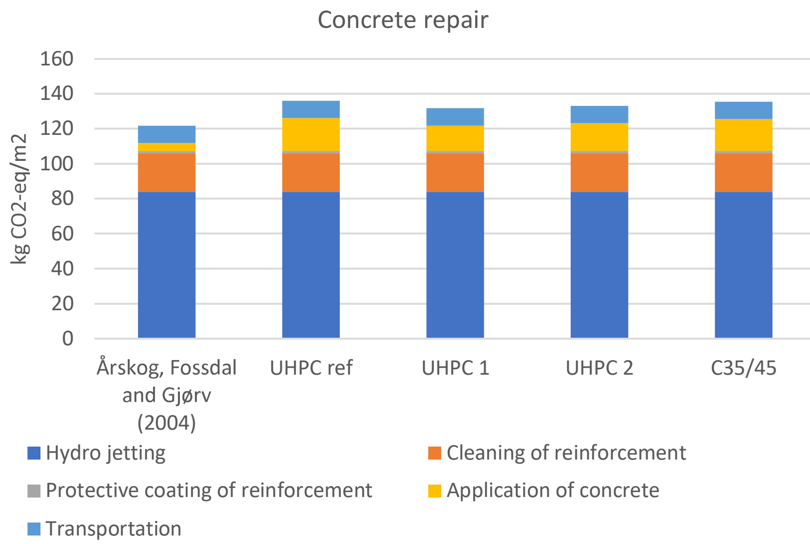

Figure 5-4 Climate impact of repairing a 50 mm cover

5.2 LCC

This study is limited to Swedish geographical boundary and to the estimated average repair costs by STA. It does not include user and societal costs due to traffic disturbance and accidents indirectly caused by the repair activities. Additionally, the analysis is made on 1 m^2 of repair area.

The cost estimation in this study was based on material costs only. For the purpose of comparing concrete types it can be assumed that all types have the same additional costs such as energy, labor and profit. Transport costs have not been considered. According to a STA calculation, the average repair cost for a pier with a repair depth at 30 to 70 mm is 10300 SEK/m^2. This cost does not include traffic disturbance and societal costs. This cost is valid for traditional concrete. Except for the repair cost the traffic and societal costs have a big impact on the total life cycle costs. This is, however, not included in this study.

5.2.1 Per cubic meter

The costs for 1 m^3 of the chosen concrete types is shown in Figure 5-5. The steel fibers are very costly and stands for the biggest cost impact. Other large contributions are due to the additives, quartz fibers, silica fume and fly ash. Here the transport distance plays a big role.

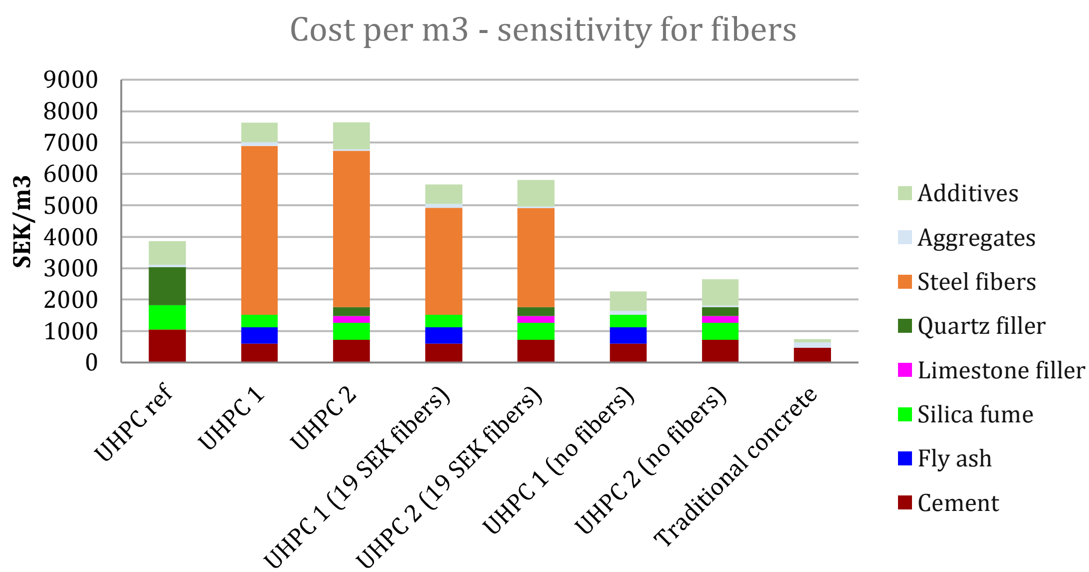

Figure 5-5 Material costs for producing 1 m³ of concrete, including a price sensitivity of steel fibers.

5.2.2 *Costs for concrete cover*

When considering the cover thickness needed to reach a 100-year service life the UHPC concrete is still more costly than conventional concrete. **Fel! Hittar inte referenskälla.**5-7 shows the cost per m² of concrete cover for the different concrete types.

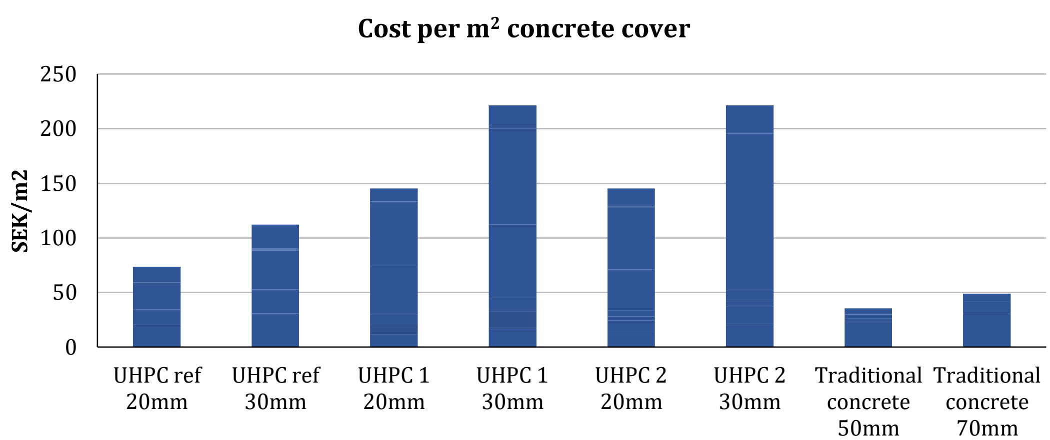

Figure 5-6 Costs per square meter of concrete cover for the chosen concrete types

The hot-spots for the concrete costs are the steel fibers. An additional analysis was therefore performed to investigate the costs without fibers. Figure 5-5-8 shows the costs for each concrete for different cover depths per 1 m². It can be noted that without fibers, both developed UHPC alternatives have a lower cost than the traditional UHPC. Although the costs are still higher than for traditional concrete the figures are now quite close. This shows that there is an improvement potential for UHPC.

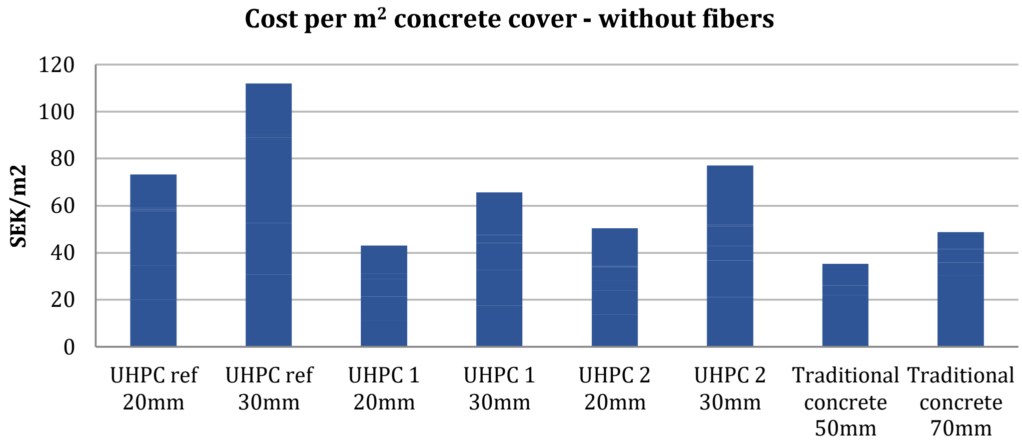

Figure 5-8. Costs per square meter of concrete cover without steel fibers for UHPC 1 and UHPC 2.

5.2.3 Considering repair costs

Since it is unknown when a repair might be needed the repair costs are shown in a graph which shows the NPV in relation to timing (Figure 5-). The cost for repairing 1 m2 of pier by using UHPC could be assumed to cost an additional 110 SEK, which is equal to the difference in material cost between UHPC and C35/45. In that case it would be beneficial to use UHPC on the following conditions:

- The UHPC cover protects the reinforcement for a longer time than the traditional concrete (preferable 100 years)
- The pier needs to be repaired within 100 years with traditional concrete
- There are no additional costs for using UHPC

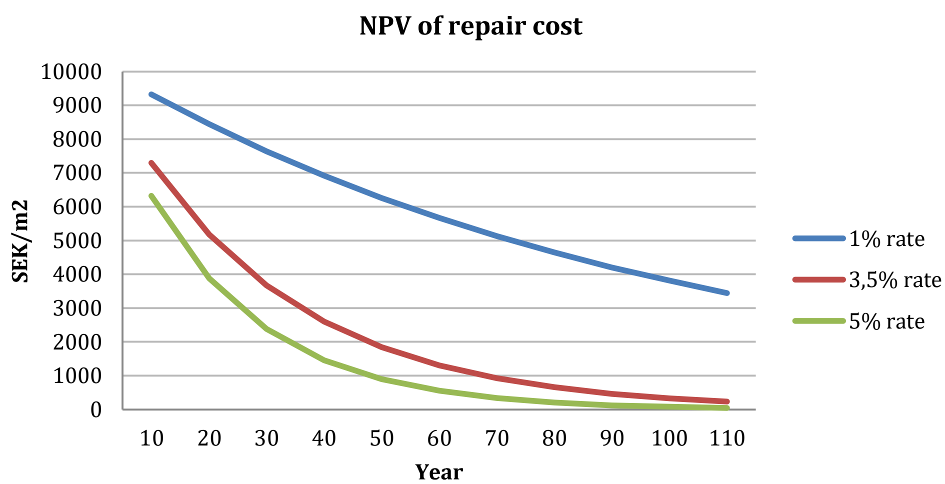

Figure 5-9. Net present value of a 50-70 mm concrete repair per square meter of bridge pier.

6 CONCLUSION/DISCUSSION

6.1 UHPC recipes properties

The developed recipes meet the prescribed requirement in fresh properties and harden state in regard to flowability, strength and durability. The materials have good fresh properties that require no vibrations but flows and fills out the frame. The compressive strength is very good and with fibers, the flexural strength is increased. The durability properties is also very good. As with any repair, the surface treatment is important. With proper surface treatment the adhesion results have been very good.

Large scale testing showed the increase in compressive and flexural strength. The cracks due to shrinkage shows the need for further development in regards to larger scale casting. The no though cracks were a good start and the no cracked samples should have even higher flexural strength. There was also signs of self-healing.

6.2 LCA/LCC

The developed UHPC mixes (UHPC 1 and UHPC 2) showed potential in lowering the greenhouse gas emissions in the following ways:

- During production of 1 m^3 of concrete due to lower clinker content compared to traditional UHPC. Additionally, further increase could be made if content of steel fibers could be reduced.
- Through a thinner concrete cover compared to traditional C 35/45.
- During the service life due to less energy intense repair activities needed.

Regarding the cost impacts the UHPC mixes showed a higher cost compared to the traditional C 35/45 concrete for both the production and amount needed for the cover thickness. However, the repair activities are more costly than the concrete itself and there is a benefit of using UHPC if it proved to have less need for repair compared to traditional concrete.

There is a benefit of using UHPC, especially in the longer-term, however there still is a potential for improvement in the production stage.

6.3 Future research

The materials shows great potential but further research is required for implementation in the field. Examples of further areas can be

- Alternatives to steel fibers or reduction of content
- Shrinkage management (possible use of shrinkage reducers)
- Pumpable UHPC
- Shotcrete UHPC
- Additional use of supplementary materials

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