THESIS FOR THE DEGREE OF DOCTOR OF PHILOSOPHY

Steel Fibres in Reinforced Concrete Structures of Complex Shapes

Structural Behaviour and Design Perspectives

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Cover:
Top left and right: A tailor-made concrete structure produced in the research project TailorCrete (www.tailorcrete.com). Photo: Michael Støvelbæck, used with courtesy of the photographer. Bottom left: Two-way slab set-up for testing.

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Abstract

In concrete structures of complex geometries, the formability of concrete is an asset. However, complex geometries introduce time-demanding form and reinforcement works. By streamlining design and production, utilising the benefits of computational tools as well as modern production technologies, the buildability of complex concrete structures can be increased. Combinined with the use of alternative reinforcement methods, the scope of unique concrete structures could be broadened. Conventionally, steel bars have been used; however, alternative reinforcement methods have been introduced. First, several reinforcement methods were studied and the focus for the remainder of the work were set on the structural behaviour of conventional reinforcement and steel fibre reinforced concrete (SFRC).

It may not be possible to apply standard idealisations to concrete structures of complex geometry. Two methods for the design of conventional reinforcement, based on linear finite element analyses, were investigated and both were found to provide a rational approach calculating the amount of reinforcement needed.

To investigate the structural behaviour of SFRC, an experimental programme was conducted. Two-way slabs with combinations of conventional reinforcement and SFRC were tested, investigating the effect from steel fibres on load redistribution. To provoke redistribution after cracking, the conventional reinforcement was arranged asymmetrically, forming a weak and a strong direction. As expected, steel fibres increased the load-carrying capacity and the number of cracks. Furthermore, the steel fibres increased the portion of applied load transferred to the supports in the weak direction and contributed to evening out the load of the length of the support. Material characterisation of the SFRC was performed through both uni-axial and three-point bending tests. A numerical approach was successfully utilised to relate the two test methods to each other.

Analytical and numerical analyses of both beams and slabs were conducted, and the results were compared with experiments. In these cases, the additional capacity provided by the steel fibres was observed both experimentally and in numerical analysis. Depending on the interpretation of analytical proposal in Model Code 2010, the load-carrying capacity was either underestimated or rather accurately estimated.

Through a combination of experimental, numerical and analytical work on both structural and material levels, this thesis contributes to an improved understanding for the structural use of SFRC, especially in structures of complex geometry.

Keywords: Concrete structres, Geometrically complex structures, Reinforcement alternatives, Steel fibre reinforcement, Rational design

Stålfiberarmerade betongkonstruktioner med komplexa geometrier
Mekaniskt verkningssätt och dimensionering
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Sammanfattning

I skräddarsydda betongkonstruktioner, dvs. betongkonstruktioner anpassade till en oregelbunden geometri, utnyttjas betongens formbarhet. Komplicerade geometrier medför dock tidskrävande form- och armeringsarbeten. Med moderna beräkningsverktyg för konstruktion och automatiserad produktion, kan byggbarheten av komplexa byggnadsverk öka. I kombination med alternativa armeringsmetoder kan det möjliggöra att unika betongkonstruktioner kan uppföras i större utsträckning än idag. Inledningsvis studerades flera olika armeringsmetoder i en litteraturstudie och forskningen inriktades därefter på konventionell armering och stålfiber.

Vanliga konstruktionstekniska förenklingar kan inte alltid appliceras på geometriskt komplexa konstruktioner. Finita element modeller kan beskriva godtyckliga geometrier, därför undersöktes två beräkningsmodeller för konventionell armering, baserade på finit element analys. Båda visade sig vara lämpliga att använda som ett led i en rationell dimensioneringsprocess.

En försöksserie utfördes för att undersöka stålfiberbetongens mekaniska egenskaper. Plattor armerade med konventionell armering och stålfibrer provades, för att undersöka stålfibernas effekt på lastomfördelning efter uppsprickning. För att tvinga fram omfördelning armerades plattorna osymmetriskt, så att en svag och en vek riktning uppstod. Resultaten påvisade att stålfibrerna bidrog både till plattornas lastkapacitet, och en jämnare lastfördelning både mellan vek och styv riktning, samt över stödlängderna. Stålfiberbetongens materialegenskaper provades både i enaxiella dragförsök och i balkar i trepunktsböjning. En numerisk metod användes för att relatera resultaten från de båda försöksmetoderna till varandra.

Stålfiberarmerade balkar och plattor analyserades med både numeriska och analytiska metoder och resultaten jämfördes med försöksresultat. I de studerade fallen kunde stålfiberarmeringens tillskott kvantifieras väl med numeriska metoder. Beroende på hur den analytiska metoden, föreslagen i Model Code 2010, tillämpades, underskattades lastkapaciteten eller stämde relativt väl.

Genom att kombinera experimentellt arbete med numeriska och analytiska metoder, både på struktur- och materialnivå, bidrar denna avhandling till en ökad förståelse gällande stålfiberbetong i bärande konstruktioner, särskilt i geometriskt komplexa tillämpningar.

Nyckelord: Betongkonstruktioner, Geometriskt komplexa konstruktioner, Armeringsmetoder, Stålfiberbetong, Rationell dimensionering.

Preface

The research for this thesis was carried out at the Division of Structural Engineering at Chalmers University of Technology from 2009 until 2014. The major part of the research was funded by the European Community's Seventh Framework Programme under grant agreement NMP2-LA-2009-228663 (TailorCrete). More information on the TailorCrete research project can be found at www.tailorcrete.com. I would like to acknowledge all project partners who made my research, resulting in this thesis, stimulating.

I deeply appreciate and respect my supervisors, who all deserves endless praise. My main supervisor, Professor Karin Lundgren, contributed to my embarking on this research journey. I could not think of a better supervisor: always providing her honest opinion and support, with an endless patience. Assistant Professor Rasmus Rempling deserves gratitude, not only for being a great co-worker and friend, but also for his ability to provide a fresh angle on problems. In the beginning, Professor Kent Gylltoft played a major role in my work, providing valuable advice, often based on his notable experience. It might sound like a cliché, but it is most certainly true: without all of you it would have been impossible to complete this thesis.

Furthermore, I am thankful to all co-authors working with the included papers, contributing their interest and expertise. Although all have been valuable, Adjunct Professor Ingemar Löfgren deserves a special acknowledgement for providing comments and insights both in developing the test set-up and in spreparing this thesis. Many thanks to our laboratory technician Lars Wahlström for bearing with me during the experimental work. I am also grateful to all my other colleagues, past and present, at the Division of Structural Engineering. The language editing work by Gunilla Ramell was highly appreciated.

Many thanks go to my friends and family, especially my parents. You have all contributed to this effort more than you realize. Last, but certainly not least: They say that "behind every successful man there is a woman". In my opinion that saying is rather outdated and untrue. The woman in this case is my wonderful wife and she is not behind, but rather always side-by-side. Thank you for all the joy you bring and for helping me keeping life in balance.

Gothenburg, April, 2014
David Fall

Appended papers

This thesis consists of an extended summary and the following appended papers:

Paper A	"Reinforcing tailor-made concrete structures: Alternatives and challenges" D. Fall, K. Lundgren, R. Rempling and K. Gylltoft. (2012) *Engineering Structures*, Vol.44, pp.372-378
Paper B	"Non-linear Finite Element Analysis of Steel Fibre Reinforced Beams with Conventional Reinforcement" D. Fall, R. Rempling, A. Jansson, K. Lundgren, and K. Gylltoft. (2012) In: *BEFIB2012 – Fibre reinforced concrete*, Eds: Joaquim Barros et al.
Paper C	"Two-way slabs: Experimental Investigation of Plastic Redistributions in Steel Fibre Reinforced Concrete" D. Fall, S. Jiangpeng, R. Rempling, K. Lundgren, and K. Zandi. *Submitted to Engineering Structures.*
Paper D	"Material characterization of concrete reinforced with double end-hooked steel fibres" D. Fall, R. Rempling, I. Löfgren, M. Flansbjer and K. Lundgren. *Submitted to Magazine of Concrete Research.*

Author's contribution to appended papers

The contributions of the present author to the appended papers are described below:

Paper A Responsible for the planning, most of the research and writing. Conducted the literature study, implemented the design method discussed and calculated the example presented.

Paper B Responsible for writing most sections and all numerical modelling. The experimental work in this paper was carried out by Dr. Anette Jansson.

Paper C Responsible for the development of the test-set up, as well as planning and executing the experimental series and the evaluation of results. Planned, coordinated and wrote most sections.

Paper D Conducted the numerical modelling. Coordinated all material testing and executed the three-point bending tests. The uni-axial tension tests were performed and put into typing by Dr. Mathias Flansbjer. Planned and coordinated the writing, and wrote most sections.

Other publications related to this thesis

In addition to the appended papers, the author of this thesis has also contributed to the following publications:

Licentiate thesis

Fall, D. (2011): *Reinforcement in Tailor-Made Concrete Structures.* Licentiate Thesis. Depertment of Civil and Environmental Engineering, Division of Structural Engineering, Concrete Structures, Chalmers University of Technology, Publication no. Lic 2011:5, Göteborg, Sweden.

Journal papers

Rempling, R., Fall, D. and Lundgren, K. (2014): "Principles of integrated design: parametric models, creative space and linked knowledge", *Submitted to Automation in Construction.*

Williams Portal, N., Nyholm Thrane, L., Lundgren, K. and Fall, D. (2014): "Flexural Behaviour of Textile Reinforced Concrete (TRC)", *Submitted to Cement and Concrete Composites.*

Jiangpeng, S., Fall, D., Plos, M., Zandi, K. and Lundgren, K. (2014): "Analysis of Tested Two-way RC Slabs with Varying Modelling Choices", *To be submitted for International Publication.*

Conference papers

Fall, D., Lundgren, K. and Gylltoft, K. (2011): "Reinforcement in Tailor-Made Concrete Structures", *Proceedings of XXI Nordic Concrete Research Symposium*, June 2011, Hämeenlinna, Finland.

Fall, D., Lundgren, K., Rempling, R. and Gylltoft, K. (2011): "Structural Design of Reinforcement in Tailor-Made Concrete Structures", *Proceedings of the 7^{th} Central European Congress on Concrete Engineering*, September 2011, Balatonfüred, Hungary.

Fall, D., Lundgren, K. and Rempling, R. (2013): "Modelling Cracking and Bending Failure of SFRC-beams with Conventional Reinforcement", *FraMCoS-8, VIII International Conference on Fracture Mechanics of Concrete and Concrete Structures*, March 2013, Toledo, Spain.

Fall, D., Rempling, R. and Lundgren, K. (2013): "Experiments on Fibre Reinforced Concrete Two-way Slabs", *7th International Conference FIBRE CONCRETE 2013*, September 2013, Prague, Czech Republic.

Rempling, R., Fall, D. and Lundgren, K. (2013): "Three-point bending and Uni-axial Tension Tests of Double Hook-end Fibers", *7th International Conference FIBRE CONCRETE 2013*, September 2013, Prague, Czech Republic.

Jiangpeng, S., Fall, D., Plos, M., Zandi, K. and Lundgren, K. (2014): "Finite Element Modelling of Two-way RC Slabs with Varying Modelling Choices", *Accepted to the 107th fib International PhD Symposium in Civil Engineering*, July 2014, Québec, Canada.

Technical Report

Fall, D. and Nielsen, C. (2010): *Concrete reinforcement solutions*, Report 2010:7, Division of Structural Engineering, Department of Civil and Environmental Engineering, Chalmers University of Technology, Gothenburg, Sweden.

Popular scientific papers

Fall, D., Lundgren, K. and Gylltoft, K. (2010): "Skräddarsydda betongkonstruktioner", *Bygg & Teknik* 7:20, popular scientific in Swedish.

Fall, D., Williams Portal, N. and Lundgren, K. (2013): "Formbarare betong med ny armering", *Tidskriften Betong*, (No.5), popular scientific in Swedish.

Contents

1 Introduction

1.1 Background

Concrete is a brittle material; the tensile strength is low in relation to the compressive strength. To compensate for the low tensile strength, reinforcement is normally used in load carrying structures. During the past century, reinforcement has typically consisted of steel bars, placed in the formwork prior to the casting of the concrete. Advances in material technology has made it possible to utilise a wide range of alternatives to the conventional steel bar.

In contrast to the production processes of many other building materials, the casting of concrete allows for the production of concrete structures in complex shapes. This potential for architectural freedom is one of the most attractive features of this material, as exemplified in Figure 1.1. However, complex geometries introduce complex and time-demanding formwork, complicated reinforcement layouts, with the result depends on good craftsmanship. In brief, any shape is possible; however, simple and straight is the most cost-effective. By streamlining design and production, utilizing the benefits of advanced computational tools as well as modern automated production technology the prices of uniquely shaped concrete structures can be reduced. Combining such industrial thinking with the use of alternative reinforcement methods, the scope of unique concrete structures could be broader than prestigious projects. This development defines the term "tailor-made concrete structures": concrete structures of complex shapes made possible by utilizing modern digital fabrication methods.

Conventional reinforcement, alone or in combination with fibre reinforcement, has the potential for application in tailor-made concrete structures. During the past decades, it have been proven that short fibres, mixed into the fresh concrete, can add ductility to the otherwise brittle material and are particularly interesting for applications in which conventional reinforcement is hard to produce (e.g. Domingo *et al.* 2004).

1.2 Aim and objectives

The aim was to contribute to the development of tailor-made concrete structures by proposing reinforcement solutions suitable to such structures of complex shapes. Further research needs, hindering the application of the selected reinforcement solutions, were identified and addressed.

(a) *L'Oceanogràfic, Valencia, Spain*

(b) *Shell House, Nagano Japan*

(c) *TailorCrete prototype, Aarhus, Denmark*

Figure 1.1: *Examples of geometrically complex concrete structures. Photo (a) by David Iliff. Licensed under Creative Commons (CC-BY-SA 3.0). Photo (b) by Artechnic and photo (c) by Michael Støvelbæck, both used with permission.*

The objectives of the work presented in this thesis were the following:

- Choosing the most suitable reinforcement solutions for tailor-made concrete structures.
- Further investigating the selected reinforcement solutions, identifying and addressing the research needs associated with structural application, e.g. problems in design or gaps in knowledge of the structural behaviour.

Based on the objectives above, it was concluded that conventional reinforcement and steel fibre reinforcement were suitable options. The following objectives were then formulated:

- To suggest a rational design process for concrete structures of complex geometry.
- To investigate the applicability of analytical and numerical methods to steel fibre reinforced concrete.

As these objectives were met, further research needs were identified, resulting in additional objectives:

- To study the effect of steel fibre reinforcement on load distribution in statically indeterminate systems.
- To further investigate the applicability of analytical and numerical methods to reinforced concrete structures with steel fibres, expanding the study to statically indeterminate structures.

1.3 Scientific approach and methodology

The fundamental research strategy is visualised, somewhat simplified, in Figure 1.2. The direction of research was chosen during an initial phase. Within the scope chosen, research problems or hypotheses were formulated, research was conducted followed by a conclusion. In most research, conclusions tend to not only push the research frontier forwards but also raise new questions; hence, the iteration loop revolves back to the problem formulation. The research strategy described might seem trivial on this general level, but is presented as applied in Figure 1.3.

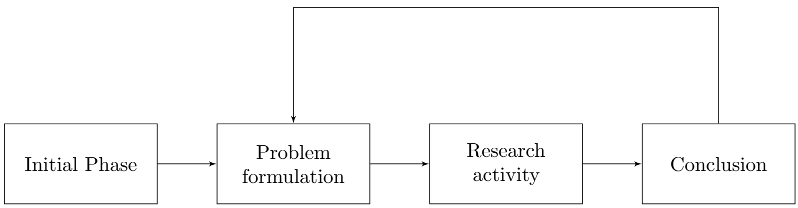


Figure 1.2: *Fundamental scientific approach.*

In the work resulting in this thesis, research activities of different nature were performed: literature reviews, experiments, and numerical as well as analytical studies were combined. Applying the fundamental approach presented above to the study at hand, the initial phase consisted of a thorough literature review and evaluation of alternative reinforcement types considering the parameters given in Section 2.1. Based on this literature review, it was decided to mainly focus on conventional steel reinforcement and steel fibre reinforcement, leading to two parallel paths (Figure 1.3, Iteration 1).

In the continued work with rational design of conventional reinforcement, both a literature study and an analytical approach were utilized. Working with steel fibres in reinforced concrete beams, numerical and analytical methods were applied and results were compared with experimental results. Addressing the research needs identified in the first iteration, an experimental programme was initiated. Again, results were compared with numerical as well as analytical methods.

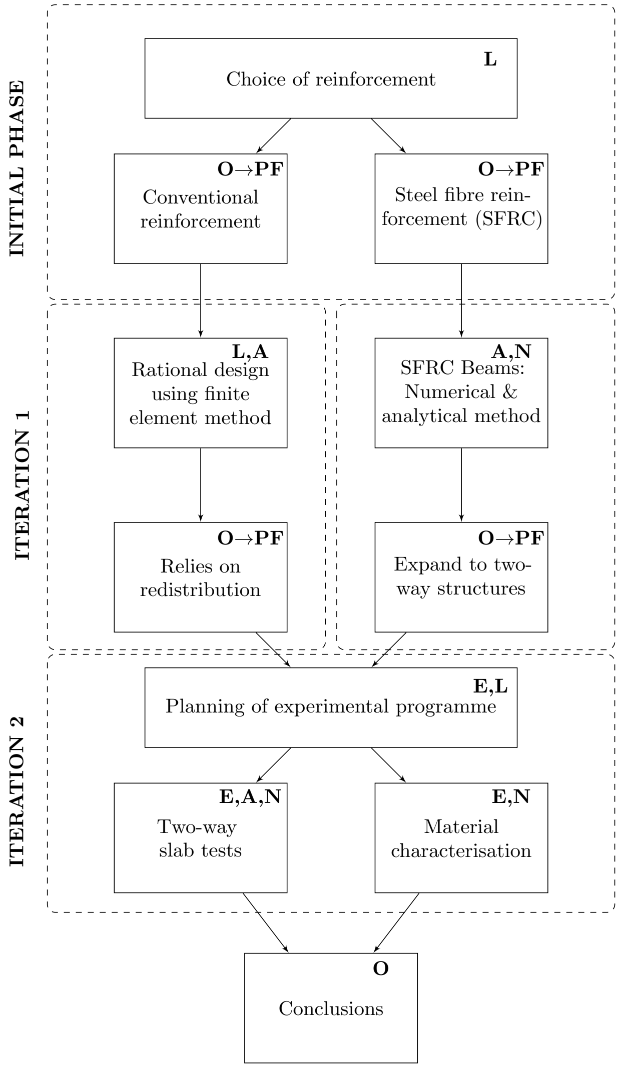


Figure 1.3: *Description of work. Compare the iterative process in Figure 1.2. The letters in the top right corner of each box denotes the used methodologies within each research activity:* ***L*** *- Literature study,* ***N*** *- Numerical approach,* ***A*** *- Analytical approach and* ***E*** *- Experimental approach. The letter* ***O*** *denotes that the box describes an outcome and* ***O→PF*** *is an outcome leading to a new problem formulation.*

1.4 Scope and limitations

An overview of the available reinforcement methods, documented by the research community, is provided and discussed. The discussion and evaluation of these methods concentrate on applications for buildings, although the concept could be applied to any structure. Following the evaluation, the work was limited to the reinforcement types chosen, i.e. conventional reinforcement and steel fibre reinforcement. Work conducted on rational design methods for conventional reinforcement was limited to design methods previously reported in literature. The potential application of the methods reviewed was limited to shell-like structures (c.f. Figure 1.1). Moreover, the initial study on the modelling of steel fibre reinforced structures was limited to beams of varying fibre content.

The experimental programme during the latter stage of research was conducted using two types of concrete: plain concrete and steel fibre reinforced concrete. Neither of the concrete compositions was varied. Only one fibre type and one fibre content were tested. Furthermore, the geometry of the slabs tested was kept constant. Material testing was limited to compression tests and two test methods characterising the tensile behaviour: three-point bending and uni-axial tensile tests. When comparing experimental results with design codes, only Model Code 2010 was used, as it can be considered the research frontier of design standards.

1.5 Research significance

The experimental programme on two-way slabs features several unique contributions to the scientific community. The test set-up derived for slab testing provides information on the load distribution, both with regard to the load-carrying directions and distribution along the supports, providing important information on the structural behaviour, both with and without steel fibres. The effect of steel fibres on the load distribution in statically indeterminate structures has, to the author's knowledge, not been previously studied. In addition, the double hooked-end steel fibre used in the study constitutes a new generation of structural fibre. No prior studies on the behaviour of this fibre type have been found in literature. Concluding the experimental series, the study contributes to an improved the understanding of the structural behaviour of SFRC, indicating the great potential of the material.

This work contributed to the development of tailor-made concrete structures, i.e. geometrically complex concrete structures made possible by the implementation of automated concrete production, by addressing the reinforcement in such structures. The first part of the work consists of a literature review. To the author's knowledge, no other work provides similar overview. Furthermore, the research was conducted from a viewpoint rarely considered: weighing design, production and structural aspects. Such an holistic approach to structural design can potentially rationalise and ease the everyday work of structural engineers.

1.6 Outline of thesis

The thesis consists of seven chapters in an introductory part and four appended papers. The introductory part summarises the papers and some additions.

In Chapter 2, different reinforcement alternatives are presented. Furthermore, they are evaluated with regard to different properties discussed in the chapter. The chapter is an expansion of the first section of Paper A. Following this effort, it was decided to focus further research efforts to conventional reinforcement and steel fibre reinforced concrete.

Rational design approaches are discussed in Chapter 3. Furthermore, the entire design process, from architectural model to reinforcement layout, is discussed. The need to use redistribution in design was acknowledged. This chapter summarises the second part of Paper A.

The tests performed are presented and the results discussed in Chapters 4 and 6, summarising Paper D and Paper C, respectively. Chapter 4 treats material characterisation by uni-axial and three-point bending tests; furthermore, the relation between the results is investigated utilising a numerical approach. In Chapter 6, focus is aimed at the slabs tested. The development of the test set-up is discussed, followed by the results. Furthermore, the results are compared with design calculations and the results from numerical analyses.

In Chapter 5, numerical modelling and analytical design methods for steel fibre reinforced concrete are discussed. Results are compared with experimental results, concluding that the code proposal, as applied, considered underestimated the load-carrying capacity while the numerical approach more accurately described the structural response. Major parts of the chapter is a summary of Paper B: however, the analytical calculation made in accordance with Model Code 2010 expands the scope.

Finally, the conclusions are presented and suggestions for future research are provided in Chapter 7.

2 Reinforcement for tailor-made concrete structures

During the past century, steel bars have been used to provide concrete structures with tensile strength and ductility in instances when the concrete itself was not sufficient. In the past 25 years, several alternatives have been developed.

In this chapter, reinforcement alternatives are reviewed in the context of concrete structures of complex geometry, to be produced utilizing the modern tools of automation. The properties affecting the applicability to tailor-made concrete structures are discussed in Section 2.1. Potential reinforcement solutions are presented in Section 2.2. A more detailed description of the literature study performed can be found in the report by Fall and Nielsen (2010). Finally, advantages and disadvantages identified are presented in Section 2.3 along with the choices made concerning the focus of the continued work.

2.1 Evaluation properties

Considering reinforced methods to be used in automated production and complex geometries, a number of properties are of particular interest. The properties that were considered evaluating advantages and disadvantages of reinforcement solutions are described in this section.

Production properties: A keystone in this work is the aim of producing unique concrete structures with greater flexibility by using robotics and unconventional formwork. This goal radically affects the choice and design of the reinforcement. When the reinforcement solution is evaluated, it is important to consider whether it can be produced in arbitrary shapes or not. Furthermore, an evaluation has to be made to determine whether the complete reinforcement solution can be produced without any manual operations.

Mechanical properties: Mechanical properties, such as tensile strength, composite toughness and composite flexural behaviour, have to be taken into account while evaluating reinforcement solutions. The tensile strength is relatively weak in the concrete itself, thus making it important for the reinforcement to provide the composite with a sufficient amount of tensile strength. Furthermore, the composite toughness governs, to a large extent, the post-cracking behaviour. The stiffness of the composite and the bond between the reinforcing material and the concrete, will also affect the mechanical behaviour. In addition to the analytical parameters above, it is important to evaluate whether the reinforcement solution would increase the load carrying capacity of the structure or simply control the post-cracking behaviour. Whether or not the reinforcement method can be used alone, or if another type of reinforcement providing the structural integrity would be needed, has also to be assessed.

Durability: For a structural member to fulfil its expected service life, it is vital to consider two properties, namely deterioration and fatigue resistance. The resistance to deterioration depends not only on the environment to which it will be exposed to, but also on the composition of the concrete. Fatigue resistance is not evaluated here since fatigue problems occur mainly in structures subjected to cyclic loading, e.g. bridges.

Economic properties: It would be desirable if the reinforcement solution, when widely adopted, could be produced in an economically efficient way.

Environmental sustainability: It should be taken into account whether the reinforcement is, or might be, produced from recycled materials. The conditions regarding the energy and carbon footprints embodied in the production of reinforcement are significant aspects of the environmental evaluation.

Regulations, standards and design rules: Design rules and building legislation govern the concrete solutions being implemented today. Therefore, one must consider whether new solutions are applicable or if new regulations are required.

Quality control: The ease of quality control is not the same for all reinforcement solutions. When structural reinforcement is considered, the various aspects of quality control should be taken into account.

2.2 Reinforcement alternatives

2.2.1 Conventional reinforcing steel

Conventional reinforcing steel can be applied either as bars, meshes or rolled mats usually placed on spacers in the mould before casting. During the preparations, the steel can be formed by bending within specific limits of the radius. Reinforcement modules (e.g. cages) can be prefabricated off-site in a variety of industrial production processes. However, installing the reinforcement in the formwork is time-consuming and labour intensive. Using prefabricated mats of reinforcement, rolled out on-site, allows for shorter construction time; however it is not an approach applicable in complex geometries (Simonsson, 2011). During extensive research and practical use of reinforcing steel, it has become evident that durability is an issue. To cope with these problems, certain requirements must be fulfilled when designing conventionally reinforced concrete structures, e.g. a minimum concrete cover which could restrict the geometrical freedom desired in tailor-made concrete structures.

Bjerking (1970) states that during a normal construction procedure, the reinforcement bars can be dislocated from the position intended by the designing engineer. Such damage may occur either because construction personnel normally have to walk on the reinforcement in order to ensure a good distribution of the fresh concrete, or because of the weight of the fresh concrete. If the concrete cover were reduced, a change of the structural capacity or a reduction in durability might occur. Furthermore, it is important that reinforcement bars are assembled with sufficient strength, by wires or welding, in order to prevent the dislocations described above.

In addition to reinforcement, steel can be utilised for prestressing for which the steel is tensioned either before (pretensioned) or after (post-tensioned) casting. The tensioned steel provides a compression stress in the concrete, which counteracts the tension stress induced by loading.

2.2.2 Fibre reinforced concrete (FRC)

In recent years, the use of fibre reinforced concrete (FRC) has increased. By dispersing discontinuous fibres of variable size and material, into the fresh concrete, ductile structural members can be produced in a labour efficient manner. In this study, we shall consider short fibres mixed into concrete so that they constitute a randomly oriented reinforcement system. Independently of fibre material, the fibres can be produced in a variety of shapes and lengths.

Although the first crack stress of a composite would not generally significantly increase, the post-cracking behaviour is affected in a beneficial manner as shown by e.g. Stang and Aarre (1992), Barros and Figueiras (1999) and di Prisco *et al.* (2009). Fibre reinforcement is most commonly used as secondary reinforcement, i.e. a primary reinforcement system, that provides the main structural integrity would be required. However, in some applications it has been used as the primary reinforcement, as reported by Oslejs (2008). To further extend such use of fibre reinforcement, research would be needed within many topics, e.g. an even distribution of the fibre network needs to be ensured. Within the TailorCrete project, this topic was addressed and a device was developed, monitoring the amount of steel fibre reinforcement in the fresh concrete at deployment was developed (NV Bekaert SA, 2014).

Regardless of the fibre material, the ability to control the fibre content throughout a structural member would be a technological breakthrough. Today, short discontinuous fibres are generally considered to be evenly distributed in all parts of a member, i.e. the fibre dosage is determined by the maximum tensile stress in the member. This could be optimised by using a digitally guided application of concrete with the addition of fibres in a nozzle, in combination with an FE model of the stress distribution of the member (Tepfers, 2008).

Fibres can be classified according to size. Löfgren (2005) describes an approach whereby the fibres are classified as macro-fibres when they are longer than the maximum aggregate size of the concrete, when their diameter is much greater than the cement grain size, and when the aspect ratio (length to diameter ratio) is less than 100. Fibres with a cross-section diameter of the same order as the cement grains and a length of less than the maximum aggregate size are classified as micro-fibres. Basically, macro-fibres increase the composite toughness by bridging macro-cracks, whereas micro-fibres increase resistance to micro-cracking prior to the formation of macro-cracks.

Reinforcing fibres can be made of steel, glass, various synthetic materials (such as carbon or polymer) or organic materials.

- Steel fibres have proven to provide significant post-crack ductility to the otherwise brittle concrete. This effect has been quantified in numerous studies, e.g Vandewalle (2000), Sorelli *et al.* (2006) and Michels *et al.* (2012). Steel fibres can be used as primary reinforcement, in some applications, and evidence good resistance to corrosion. Bentur and Mindess (2007) review several investigations of the durability of steel fibre reinforcement (SFRC). The results of these studies generally show that SFRC does not suffer any degradation due to corrosion. This improvement of the corrosion resistance, compared with conventional steel reinforcement bars, is likely because of the reduced crack width and the lack of electrical conductivity (short and discontinuous fibres). Furthermore, the spalling of concrete, caused by rust, is avoided as the small diameter of the fibres generates only a minor amount of corrosion product. Nordström (2005) studied the durability of steel fibre reinforced concrete in sprayed concrete, concluding that in the relatively thin layers of sprayed concrete, corrosion needs to be considered. Although the corrosion of steel fibre has a limited effect on structural properties in most applications, as reported by Granju and Balouch (2005), corrosion spots on the surface might affect aesthetics. Moreover, in some countries, including Sweden, because of uncertainties about whether the steel fibres would influence corrosion of the traditional reinforcement, the use of steel fibre reinforcement in combination with conventional steel reinforcement is restricted for some applications. Gil Berrocal *et al.* (2013) present a literature study on the subject and concludes that the results presented are inconsistent, see e.g. Roque *et al.* (2009), Grubb *et al.* (2007) and Mihashi *et al.* (2011). Furthermore, steel fibres have been proven to reduce crack widths because of the shrinkage in, for example, concrete overlays (Carlswärd, 2006).

- Glass fibres are most commonly used in thin, non-structural, concrete elements, e.g. façade elements. The tensile strength of glass fibres is very high; however, it may be heavily influenced by deterioration. According to Bentur and Mindess (2007), the strength of a fully aged glass fibre reinforced concrete, if exposed to an outdoor environment, is reduced to 40% of the initial strength, whereas the strain capacity is reduced to about 20% of the initial capacity. The two main mechanisms responsible for this deterioration are chemical attack and the formation of hydration products (mainly calcium hydroxide) between the filaments; these mechanisms lead to a reduction in strength and embrittlement, respectively. The use of alkali resistant glass (AR glass) limits the effect of chemical attack but does not fully prevent it. In American Concrete Institute Committee 544 (2002) and Marikunte *et al.* (1997), it is suggested that such effects might be reduced by the addition of polymer solids or pozzolan metakaolin into the concrete mix.

- Synthetic fibres can be produced from several materials with widely differing properties, e.g. polyethylene (PE), prolypropylene (PP), acrylics (PAN), polyvinyl acetate (PVA), polyamides (PA), aramid (Kevlar), polyester (PES) or carbon. According to the American Concrete Institute Committee 544 (2002), the widest current use of synthetic fibres is in flat slab applications, for which the main purpose is to control bleeding and plastic shrinkage. For such applications, fibres with a relatively low modulus, such as polypropylene and polyethylene, are used and the fibre volume content would typically be only about 0.1% by volume. Fibres added for the purpose of controlling plastic cracking are normally designated micro-fibres, with a diameter of around 40 to 100 μm. According to Naaman *et al.* (2005), smaller diameters yield better performance.

- Natural fibres are interesting mainly due to their low cost. Cellulose fibres from bamboo or sugar cane can be used as well as other natural fibres such as jute, sisal or coconut fibres (coir). As the organic fibres in their natural form, contrary to many synthetic fibres, are hygroscopic, the fibre properties are more widely affected by changes in moisture content than fibres of any other material, which may cause swelling, shrinkage or rot. Depending on the fibre material, the exposure to alkali environments causes varying amounts of strength degradation, ranging from 16% of the initial tensile strength (sisal) up to 50% (coir), according to Ramakrishna and Sundararajan (2005).

2.2.3 Fibre reinforced polymer (FRP)

In the past decade, the use of fibre reinforced polymer (FRP) has increased. By incorporating continuous fibres of aramid, carbon or glass into polymer matrix, a reinforcing composite with unique properties can be obtained. According to Dejke (2001), the properties depend on the fibre type and matrix material, but the composite generally has a lower weight, a lower modulus of elasticity and a higher strength than steel. The low modulus of elasticity often makes requirements in the serviceability limit state decisive, according to Swedish Concrete Association (2002). In Benmokrane *et al.* (1995) polyester, epoxy and vinyl ester are mentioned as examples of matrix materials.

Fibre reinforced polymer can compete with steel mainly in applications in which the steel risks corrosion to a higher extent than normally. In such applications, FRP can be utilised in several product types, including bars, cables, wraps, profiles or plates and they can be cast into concrete or used as permanent formwork. In the present study, FRP bars were the main consideration.

The FRP bars are, according to Bakis *et al.* (2002), typically produced through pultrusion or the closely connected production method of pull-forming. Simply described, the continuous fibres are impregnated with the matrix polymer and then pulled through a forming die. The FRP bar can then be bent or cut. Once the polymer material is hardened, the bar cannot be formed. The bonding between the bar and the surrounding concrete can be improved by moulding ribs in the bar, by winding one or more fibre tows around it, or by bonding a finely grained aggregate (eg. sand) onto the surface of the bar.

The FRP bars, in contrast to conventional reinforcement, do not corrode; however, other degradation mechanisms still limit the durability. Dejke (2001) mentions that a concrete pore solution might initiate deterioration. Furthermore, he lists sea salt, de-icing salt, freeze-thaw cycles, UV light and fresh water as potential durability threats. In the International Federation for Structural Concrete (fib) (2007), a comprehensive overview of research performed on the durability, as well as many other aspects, of FRP is provided.

Dejke (2001) and Swedish Concrete Association (2002) state that, for FRP composites with glass or carbon fibres, equal or less creep and relaxation than "low relaxation steel" have been reported. Aramid fibre composites have shown significant creep and relaxation (eg. creep of 7% after 50 years of 40% short-term strength loading).

In general, FRP exhibits brittle fracture because the modulus of elasticity, according to Karlsson (1998), is approximately constant until fracture. The ultimate strain varies between 0.5 and 2% for carbon fibres, and between 2 and 4% for glass or aramid fibres. A more ductile behaviour could be obtained by combining some of these fibre materials within a composite (Harris *et al.* 1998). Ductility might also be provided through a combination with fibre reinforce concrete (Wang and Belarbi, 2011 and Alsayed and Alhozaimy, 1999).

Stress corrosion can be described as the combined effect of long-term stress and chemical attack. During continuous loading, micro-cracks develop in the polymer matrix, which allows aggressive chemical substances to attack the fibres within the composite, ultimately leading to failure of the fibre. According to Swedish Concrete Association (2002), glass fibre composites are highly sensitive to stress corrosion; hence, long-term loading should not exceed 25% of the short-term capacity. Although, aramid fibres are more resistant to this process, stress corrosion might still occur but is preceded by large creep deformations. Carbon fibres are not prone to failure because of to stress corrosion.

At elevated temperatures, varying behaviours have been observed for the fibre materials. Dejke (2001) reports that glass and carbon fibres retain most of their tensile strength, whereas aramid fibres will have lost approximately 75% of their strength at 250°C compared with 60°C. The composite behaviour under fire needs to be carefully considered during the design process as FRP systems are sensitive to fire (Bisby *et al.* 2005).

2.2.4 Textile reinforcement (TRC)

Textile reinforced concrete (TRC) utilises multi-axial fabrics (e.g. nets or mats) in order to improve composite tensile strength and ductility. These technical textiles provide a more efficient reinforcement solution than short randomly distributed fibres; however, they are somewhat more complex to produce and install. Textile reinforcement is also a common solution for the reinforcement of façade solutions either in panel sheets or in self-supporting sandwich elements (see e.g. Hegger *et al.* 2011 and Malaga *et al.* 2012).

Typical materials used in TRC are fibres of AR glass, carbon or aramid; however, thin steel wire or polymer fibres can also be used. Technical textiles from basalt fibres have also been used as textile reinforcement (Wei *et al.* 2010). Although fabrics can be composed in numerous ways, not all are suitable for use in concrete applications. Important fabric properties include displacement stability during application and a sufficiently open textile structure. Keeping these properties in mind, Brameshuber (2006) states that warp knitting with the insertion of reinforcing threads is better suited to concrete reinforcement than braiding. As there is no need for concrete cover with regard to corrosion, textile reinforcement is particularly suitable for thin structures (Orlowsky and Raupach, 2011).

The technical textiles can be produced in such a way that several thread systems with different orientations are incorporated in order to obtain higher strengths in multiple directions. Special machinery (a double needle bar Raschel machine) can produce two fabrics simultaneously and link them together to form a spacer warp knit. The spacing between the fabrics is normally between 15 and 100 mm and can be varied during the production to form irregular shapes. Furthermore, the fabrics can be produced independently, i.e. different materials, mesh patterns and weft thread densities can be used.

The least complex production method of textile composites is hand-lay up. Briefly, the inside of a mould is covered by the textile prior to casting. The production method is widely used in other industries, for example in manufacturing boat hulls, wind-turbine blades or aircraft wings. The main benefit of this method in concrete applications is that a fairly high fibre volume can be obtained. A disadvantage is the labour-intensive production, which requires some skill to keep the quality of the product within reasonable bounds. For fabric-cement laminate composites, the composite can be produced by pultrusion. The textile is first passed through a slurry infiltration chamber, then rolled between two rollers to press the slurry into the textile while forming the composite and removing excessive concrete. When using pultrusion, the workability of the concrete-mix is crucial. It has to be fluid enough for the textile to pass through, yet dense enough to ensure the concrete remains on the textile. Extrusion techniques could be used to produce fabric-cement composites. The basic principle of extrusion is that the concrete is being injected or compressed under pressure into a closed mould. These techniques were developed for short fibres, but they can also be adopted to textile reinforcement by placing the fabric inside the mould before casting (Brameshuber, 2006). Additionally, textile reinforcement are well-suited for the repair or strengthening of existing concrete structures, as exemplified by Mechtcherine (2013).

2.3 Summary and choice of reinforcement alternatives

The most distinctive properties of four reinforcement systems are presented in Table 2.1. A similar summary has been made for the different fibre materials previously described. These advantages and disadvantages were considered in evaluation.

Table 2.1: Summary of the advantages and disadvantages of four reinforcement systems.

	+	−
Conventional reinforcement steel	Provides structural integrity. Widely used, i.e. easy to implement with regard to guidelines and design codes. Thermal expansion normally equal to that of concrete. Inexpensive.	Might be difficult to produce effectively in arbitrary geometries. Specific concrete cover needed, i.e. not suitable for very thin structures
Fibre reinforcement	Can be added to concrete during mixing.	Rarely used as primary reinforcement.
Fibre reinforced polymer	Good durability with regard to corrosion. Can be used in thin concrete members.	Affected by degradation mechanisms e.g. UV-light and salts. Rare technology which can lead to high costs. Generally brittle fracture. Fixed shape once produced. Thermal expansion differing from that of concrete, leading to restraint forces. Fire resistance needs to be carefully designed.
Textile reinforcement	Can be applied in arbitrary geometries. Allows for thin structures.	Restricts the concrete mix design to a high extent. Rare technology which can lead to high costs. Production method needs development.

Table 2.2: Summary of advantages and disadvantages of five different fibre materials.

	Advantages	Disadvantages
Steel fibre reinforcement	Good mechanical behaviour. Very well developed. Inexpensive.	Authorities in some countries (e.g. Sweden) do not allow combination with conventional steel reinforcement in chloride environment. Possible corrosion spotting on surfaces.
Glass fibre reinforcement	High strength.	Strength decreases with time. Sensitive to alkali attack.
Polymer fibre reinforcement	Non-corrosive.	Creep. Elevated temperature can cause problems. Low Young's modulus.
Carbon fibre reinforcement	Alkali resistant. Dimensionally stable. High strength.	Hard to obtain a good distribution in concrete mix
Natural fibre reinforcement	Available in the developing world.	Hard to ensure fibre quality. Hygroscopic. Complex to assess durability.

This study shows that conventional steel reinforcement cannot easily be set aside when designing load carrying concrete structures, as none of the other reinforcement types discussed can provide such integrity in all applications. Furthermore, conventional reinforcement also increases the applicability of the new construction concept devised, as it is well-known and regulated by standards worldwide. However, alternative reinforcement techniques, e.g. steel fibre reinforcement, can be included to contribute additional structural integrity in terms of ductility. As previously mentioned, the production method is important to keep in mind when considering reinforcement for tailor-made concrete structures. Recent advances in automation shows that both bending and fastening of conventional reinforcement in a fully automated procedure are possible, at least in limited series (Cortsen *et al.* 2012 and Cortsen, 2013).

Based on the evaluation of reinforcement alternatives, it was decided to continue developing a reinforcement solution for load-bearing tailor-made concrete structures by utilizing both conventional reinforcement and steel fibre reinforcement, separately or in combination. Textile reinforcement might be interesting; however, limitations are set by the flexibility of the textile. It may, simply put, be impossible for robots to handle such materials.

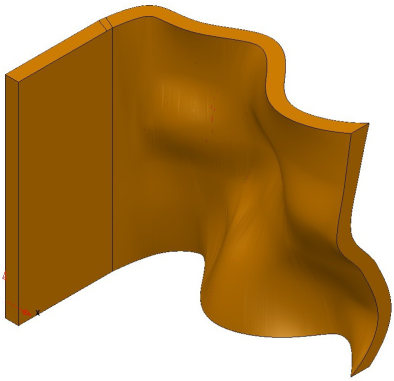
2.4 Identifying research questions

Focusing the continued work on conventional reinforcement and steel fibre reinforcement, some research needs could be identified:

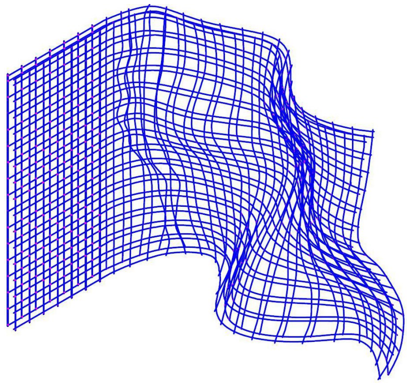
- Although conventional reinforcement has been widely used for a long time, the design of complex geometries could not be considered common knowledge among practising structural engineers. Current approaches would involve a high degree of simplification and could be further rationalized. A review of rational design methods available was needed.
- Furthermore, the flow of information in the design process is further complicated when the structure to be designed deviates from common practice. The design process needed to be analysed in terms of actors, design tools and information transfer.
- Design codes for steel fibre reinforcement are under development; however, using them is difficult and their applicability might be questioned. Code proposals need to be assessed with regard to structural members of varying kinds, both in terms of applicability and accuracy.
- Additionally, combining both reinforcement methods, structural effects, e.g. effects on the load redistributions, have not been explored. In slender concrete structures of complex shapes, the use of such redistributions play a particularly important role, as they typically are statically indeterminate. The statical indeterminacy is fundamental in design, as it makes it possible for the designer to redistribute the load.

3 Rational reinforcement design

Tailor-made concrete structures are intended to be produced by means of digital fabrications, ranging from conceptual design to production (Williams *et al.* 2011). An example of this concept can be seen in Figure 3.1. However, there are several obstacles to obtaining a seamless flow of information through the entire process, e.g. software incompatibilities and the need for idealising the structural model.



(a) *Conceptual model*



(b) *Reinforcement model*

(c) *Manufactured (unreinforced) prototype*

Figure 3.1: *Example of two models important to the design process, and the manufactured prototype. Photo: Thomas Juul Andersen, Teknologisk Institut (Danish Technological Institute).*

As conventional reinforcement was chosen to be one of the possible solutions for the tailor-made concrete structures in TailorCrete, a rational design method for conventional reinforcement is needed, suitable for structures of complex geometry. Design methods for concrete structures exist; however, a structure of complex geometry cannot necessarily be idealised with standard structural elements. Hence, designing the reinforcement requires either lengthy experience or a rational design method which also includes engineering knowledge, preferably implemented within a digital design tool.

In Paper A, the foundation for a rational design method is laid by means of a review of two design methods, originally formulated by Martí (1991) and Lourenco and Figueiras (1993). In both methods, the reinforcement design starts with a linear finite element analysis, using shell elements to describe the geometry. The eight independent stress components (n_x, n_y, n_{xy}, m_x, m_y, m_{xy}, ν_x, ν_y) acting in a shell element are introduced in Figure 3.2. For curved shell elements, the components usually become coupled as equilibrium conditions involve all stress resultants.

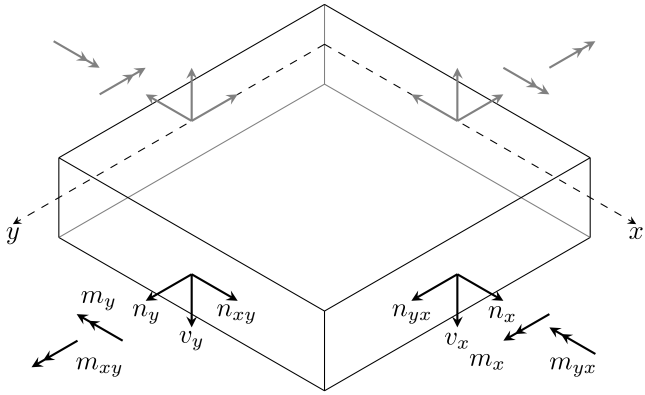


Figure 3.2: *Plane shell element with stress components: bending and membrane action.*

These methods both rely on a sandwich analogy, see Figure 3.3. For this sandwich analogy, membrane forces and bending moments are assumed to be carried in the top and bottom layers, while the core part resists shear forces.

Of the methods described, one is considerably more simplified, Martí (1991), while the other, Lourenco and Figueiras (1993), includes solving a set of six equilibrium equations, with eight unknown variables, by iteration. The simplified method relies on simplified assumptions of the lever arm and the position of the resultant steel force in the other layers, which may underestimate the lever arm, or even violate equilibrium. Furthermore, a more efficient solution in terms of reinforcement needed, is calculated using the more refined method, as shown by an example in Paper A. Hence, for future work, it would be better to use the more advanced method; using computer implementation, the simplifications of the first method are unnecessary. The method by Lourenco and Figueiras (1993) was implemented in a calculation routine. An overview of the script is shown in Figure 3.4.

By applying either of the theories described, using the results from a linear FE analysis as input, the structural engineer can calculate the amount of reinforcement needed. Such procedures rely on the theory of plastic redistributions in the cracked concrete, i.e. the

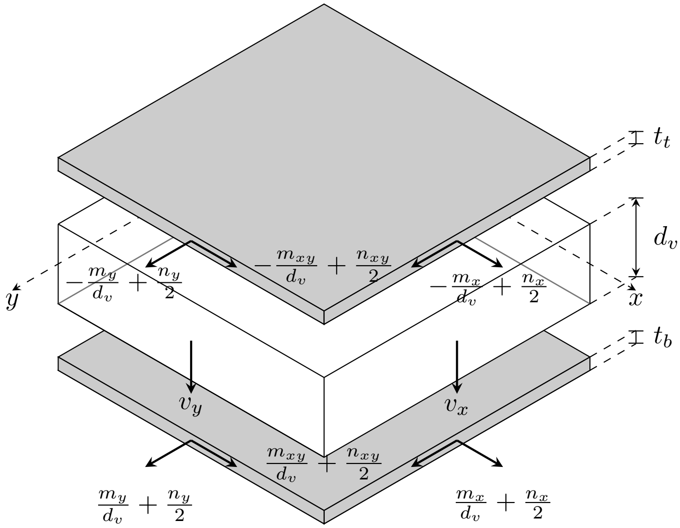


Figure 3.3: *Sandwich model with the division of the stress resultants in Fig. 3.2, see also Martí (1991).*

linear stresses are redistributed in accordance with the reinforcement design. The solution is fulfilled provided there is enough deformation capacity (ductility). The structural behaviour can be further verified by non-linear finite element analysis, for example to obtain deformation and crack patterns, see Min and Gupta (1994) or Noh (2006). Non-linear finite element analysis are increasingly used in practice, see e.g. Hallgren (2012).

Overlooking the entire design process, a design tool should ideally not only provide a mean to calculate the reinforcement amount, but also cover all stages of design. As previously mentioned, models from different actors hold differential information and a seamless flow of information is not easily achieved. The flow of information in the design process is schematically presented in Figure 3.5. Architectural models are created solely for visualisation. The model is often represented by a non-uniform rational basis spline (NURBS) geometry, which is a very common mathematical representation of a geometry. However, NURBS geometries have no parametric intelligence that can be used for computing the sectional forces. A conversion from the geometrical representation of the NURBS geometry to a numerical representation of the geometry is needed. In such conversion, the three-dimensional volumetric structure is idealised into a surface with a given thickness. Following this conversion, a model can be established in a finite element software, typically using shell elements, as exemplified here. In this stage, assumptions of the structural premises, e.g. material properties, loading and boundary conditions, have to be made. A finite element analysis provides the sectional forces from which the reinforcement amount can be calculated, using, for example, the sandwich analogy. Visualised on the model, the results provide the basis for a reinforcement layout. When a reinforcement layout has been made, the design can be verified using

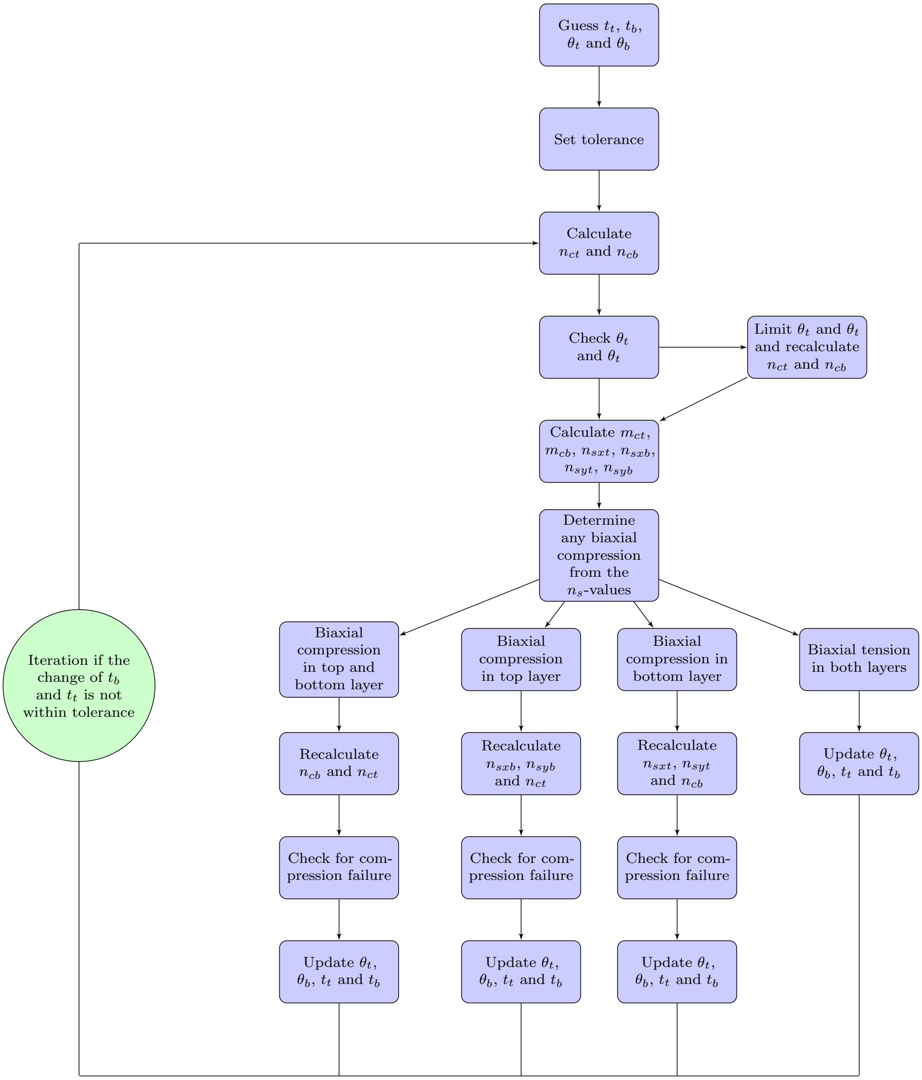


Figure 3.4: *Algorithm of the script calculating reinforcement need, based on the sandwich model of Lourenco and Figueiras (1993).*

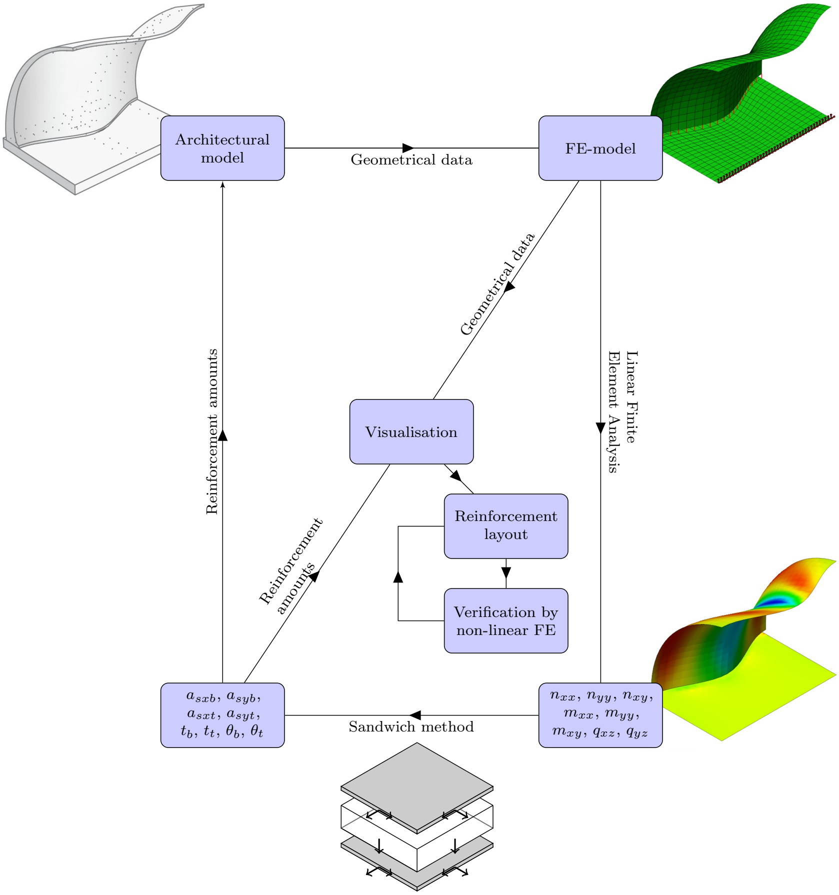


Figure 3.5: *Work-flow of the suggested design process.*

non-linear finite element analysis. Using this approach the highly non-linear behaviour of the concrete structure is taken into account; providing information on the structural behaviour including e.g. deflections and crack widths. In an iterative manner, the layout can be modified further based on the analysis result. The design process is exemplified and further discussed in Rempling *et al.* (2014).

To summarise, design methods applicable to geometrically complex concrete structures do exist. Applying them, a reinforcement amount is obtained for each part of the structure. Therefrom, a reinforcement arrangement can be obtained; however, further simplifications would be needed to make the layout feasible from a production point of view. Such simplification relies on redistributions within the structure. Redistributions are utilized in many design approaches and are further investigated through the experimental work presented in Chapter 6 and Paper C, with a focus on the effect of adding steel fibre reinforcement to the concrete.

4 Characterization of steel fibre reinforced concrete

All work with steel fibre reinforced concrete, academic studies as well as design for applications in practice, are dependent on material property characterization. Conventional (plain) concrete is generally, rightly so, assumed to have no tensile strength after cracking. Considering steel fibre reinforced concrete, the post-cracking behaviour is the most significant property. Several test methods have been proposed to characterise the brittle tensile response. Generally, the proposed methods could be classified as direct or indirect:

- **Direct** tensile methods test the tensile response, as the name implies, directly. Testing needs to be performed uni-axially. Different specimen types exist with the common feature that the crack localization is controlled by either a notch (RILEM TC 162-TDF, 2001) or a tapering (Van Vliet, 2000, Amin *et al.* 2013 and Deluce and Vecchio, 2013).

- Using **indirect** tests, parameters other than the tensile response are measured. The tensile response could then be calculated through analytical relations or numerically through inverse analysis. Indirect experiments could, for example, be based on the flexural behaviour of beams (RILEM TC 162-TDF, 2002, CEN, 2005b, ASTM C1018-97, 1997, Sorelli *et al.* 2005), panels (Bernard, 2002, ASTM C1550-05, 2005, Minelli and Plizzari, 2011), wedge-split specimens (Linsbauer and Tschegg, 1986, Brühwiler and Wittmann (1990), Löfgren, 2005), or perpendicular deformations under compression (Molins *et al.* 2009).

In Paper D, two methods, both proposed by the International Union of Laboratories and Experts in Construction Materials, Systems and Structures (RILEM), were used to characterise the concrete: a direct uni-axial tension test and an indirect three-point bending test. The test methods are proposed in RILEM TC 162-TDF (2001) and RILEM TC 162-TDF (2002), respectively.

The relation between the test methods were studied using a finite element approach. The design method proposed in Model Code 2010, International Federation for Structural Concrete (fib) (2010), is based on the three-point bending test; hence, a validated approach of relating alternative test methods to each other could be of great value. In addition to the steel fibre reinforced concrete, a mix of plain concrete was tested employing both test methods. In the following, the work is briefly presented; details are given in Paper D. The experiments presented in this section were carried out as a part of the experimental programme further presented in Chapter 6.

4.1 Uni-axial tensile tests

Uni-axial tensile testing was performed based on RILEM TC 162-TDF (2001). Six specimens were prepared and tested, both from plain and fibre reinforced concrete. The cylinders tested had a length of $L = 100$ mm and a gross diameter of $d = 100$ mm. Six cylinders were cored from larger prisms $200x200x750$ mm^3. At the mid-section of the cylinders, a 10 mm deep and 5 mm wide circumferential notch was cut.

The tests, which were displacement-controlled, were carried out in a servo-hydraulic machine with a stiff load frame. The tests were conducted using a moment stiff loading device in order to suppress rotations of the holders that might lead to bending failure. In Figure 4.1, the test set-up is shown. The lower metal fixture was glued to the cylinder with coinciding centres. The displacement was measured locally over the notch with three inductive displacement transducers. The mean value of the three displacements was used for the displacement control.

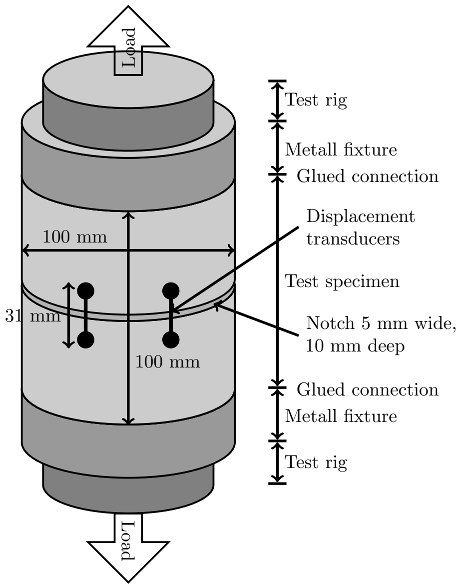


Figure 4.1: *Test set-up of uni-axial tension test.*

The results from the uni-axial tensile testing of steel fibre reinforced concrete are presented in Figure 4.2. A scatter, corresponding to the different number of fibres bridging the cracked zone (Table 4.1), can be observed. Note that three of the specimens tested (1, 3 and 4) experienced a sudden stiffness loss, before finding a new residual level. This drop coincided, in all three cases, with the formation of a second crack outside of the notch. Therefore, these specimens were excluded from the material response used in the numerical analysis, see Section 4.3.

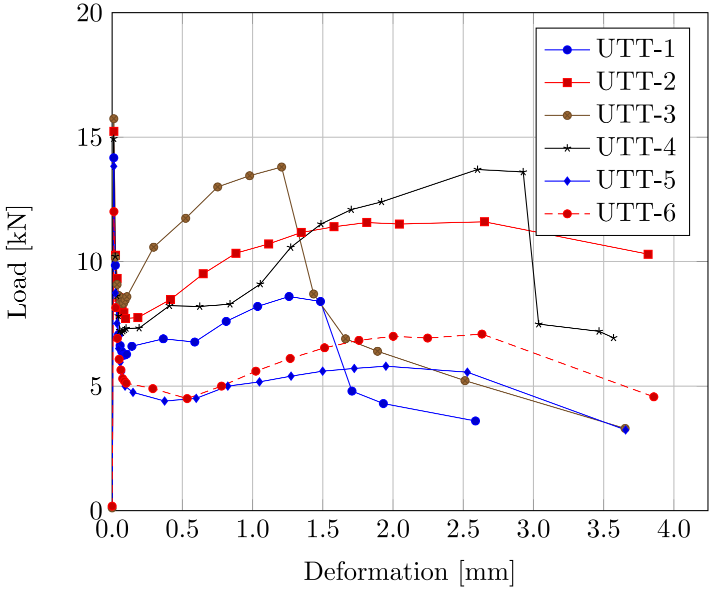


Figure 4.2: *Test set-up of uni-axial tension test.*

Table 4.1: Fibre content in the uni-axial tensile test (UTT) specimens.

Specimen	No. of fibres [fibres/cm^2]
UTT-1	0.38
UTT-2	0.34
UTT-3	0.58
UTT-4	0.56
UTT-5	0.38
UTT-6	0.38
Average	0.43
C.O.V.	24%

4.2 Three point bending tests

In addition to the uni-axial tensile testing presented in the previous section, three-point bending tests, in accordance with RILEM TC 162-TDF, 2002, were performed. The test set-up is illustrated in Figure 4.3. The beams were instrumented with two linear variable differential transformers (LVDT) measuring the deflection on the top surface and a clip gauge measuring the crack mouth opening displacement (CMOD)in the notch.

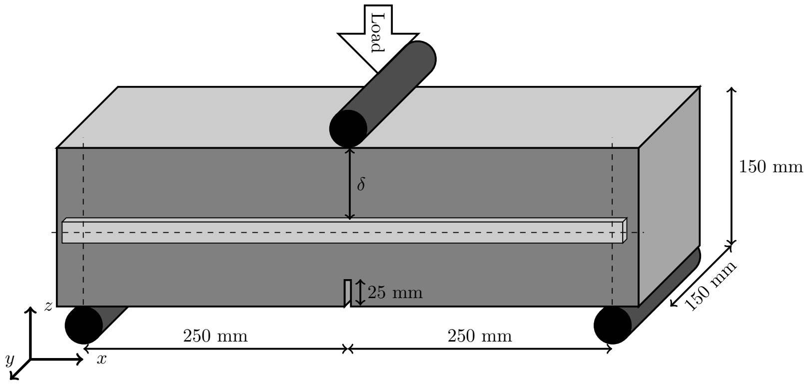


Figure 4.3: *Test set-up of three-point bending test.*

The results from the three-point bending tests are shown in Figure 4.4. The difference in the number of bridging fibres, and their position, is presented in Table 4.2. It is obvious that the amount of fibres in the fractured area significantly affects the flexural response. The residual strength is more than doubled for the specimen with a higher number of fibres in the lower section.

Table 4.2: Distribution of fibres in fractured section of the three-point bending tests, with cross-section as defined in Figure 4.5.

	No of fibres in cracked section					
Specimen	Upper	Up. mid.	Low. mid.	Lower	Total	fibres/cm^2
3PBT-1	7	25	9	8	49	0.26
3PBT-2	12	8	6	3	29	0.15
3PBT-3	14	21	5	6	46	0.25
3PBT-4	13	31	22	24	90	0.48
3PBT-5	17	22	15	15	69	0.37
3PBT-6	13	12	9	19	53	0.28
					AVG	0.30
					COV	38%

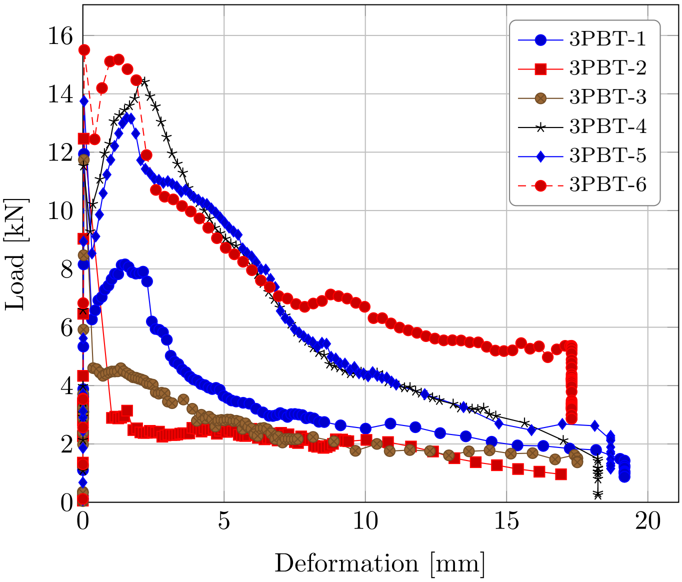


Figure 4.4: *Test set-up of uni-axial tension test.*

Upper
Upper middle
Lower middle
Lower
NOTCH

Figure 4.5: *Cracked beam section with defined zones, as used in Table 4.2.*

4.3 Comparison of test methods through numerical analysis

The two experimental approaches presented in the previous sections are not directly comparable. To facilitate such comparison, a numerical model was derived. A finite element model was established using the commercially available software DIANA (TNO DIANA, 2012). The model is schematically presented in Figure 4.6. The beam was represented by using four-node quadrilateral plane stress elements. The crack was modelled using line interface elements. Deformation was added in one node on each side of the discrete crack.

The interface elements representing the notch were assigned a multi-linear material response derived from uni-axial tension tests and compression tests. The analysis results

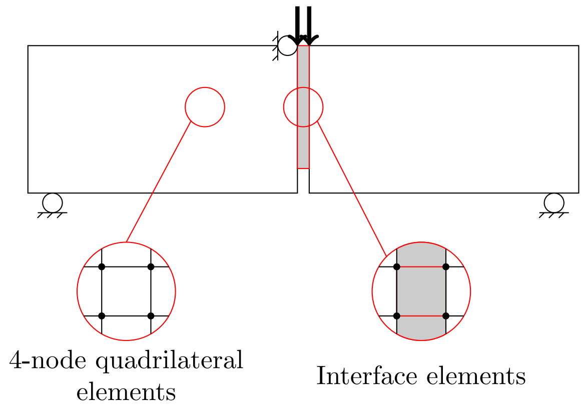


Figure 4.6: *Schematic sketch of the finite element model used in analyses.*

were assumed to be highly dependent on the tensile input, which were derived from the uni-axial tensile tests, in which significant scatter was observed. Therefore, several analyses were performed covering the range of experimental response.

In Paper D, a parametric study was performed varying not only the tensile behaviour of the interface element but also the mesh size, assumptions made for material in compression and element type. The effects observed from these variations were negligible. Furthermore, different assumptions of the tensile response at deformations larger than 4 mm were studied, affecting the results at larger deformations.

The results from the numerical analysis are presented in Figure 4.7, along with the extreme experimental results. Results are presented up to a deflection of 5 mm, constituting the range considered while evaluating design parameters. The full behaviour of the analyses is presented in Paper D.

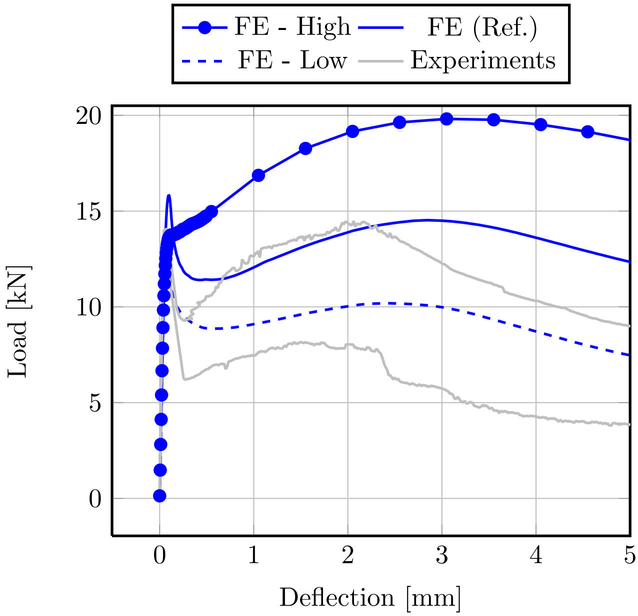


Figure 4.7: *Results from finite element analyses with varying tensile stress-deformation input assigned to the interface, based on the uni-axial tensile tests with highest and lowest strength. Results compared with the extreme three-point bending responses obtained experimentally (Sample 1 & 4).*

Comparing the experiments with the results from numerical analyses, a clear and consistent overestimation could be observed. The fibre content per unit area was considerably higher in the uni-axial specimens which is believed to be the main reason for this overestimation, cf. Tables 4.1 and 4.2. The variation was taken into account by adjusting the results with regard to a fibre efficiency factor,

$$\eta_{\mathrm{b,exp}} = \frac{N_{\mathrm{f,exp}}}{V_{\mathrm{f}}/A_{\mathrm{f}}}, \tag{4.1}$$

where $N_{\mathrm{f,exp}}$ is the number of fibres in the fractured area (per unit area), V_{f} is the fibre volume fraction and A_{f} is the cross sectional area of a fibre. The factor defines the efficiency of bridging in terms of the amount of fibres crossing a crack with respect to orientation effects and was proposed by Krenchel (1975). Löfgren (2005) proposed an adjustment, to relate test results to the theoretical fibre efficiency factor. A similar approach was chosen here, with the difference that the uni-axial test results were instead adjusted with regard to the fibre efficiency factor of the three-point bending tests:

$$\sigma_{\mathrm{b}}(w) = \sigma_{\mathrm{b,UTT}}(w)\frac{\eta_{\mathrm{b,exp,3PBT}}}{\eta_{\mathrm{b,exp,UTT}}}, \tag{4.2}$$

where $\sigma_{\mathrm{b,UTT}}(w)$ is the stress from uni-axial testing, $\eta_{\mathrm{b,exp,3PBT}}$ is the fibre efficiency factor from the three-point bending test and $\eta_{\mathrm{b,exp,UTT}}$ is that of the uni-axial tests. The results from the analyses with the modified tensile behaviour are presented in Figure 4.8, showing better agreement with the experimental results.

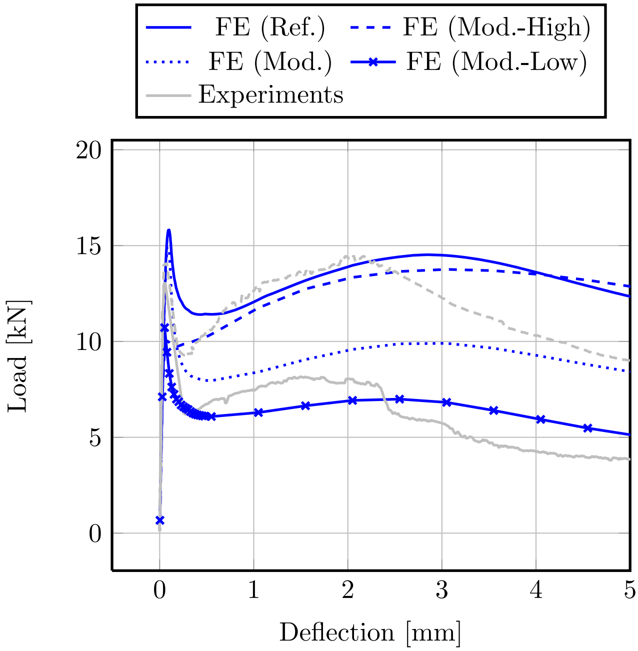


Figure 4.8: *Analysis results obtained with material properties modified to correspond to the 3PBT fibre efficiency factor. FE (Ref.) denotes the analysis performed with the unmodified average tensile response.*

To conclude, finite element modelling with the proposed approach could successfully be used to relate the test methods. The importance of documenting the number of fibres bridging fracture areas while evaluating material properties was highlighted in this study.

5 Analysis of steel fibre reinforced concrete beams

As one of the promising reinforcement concepts identified was a combination of conventional reinforcement and steel fibre reinforcement, the design and analysis methods of such structures needed to be investigated. Steel fibre reinforcement is being used increasingly, and several design codes, standards and calculation models have been proposed. Among the most established are the models proposed in Model Code 2010, International Federation for Structural Concrete (fib) (2010). However, there are also national recommendations in e.g. Germany, Spain, Italy, Australia and Norway. In addition to these recommendations, a Swedish proposal has been made (Silfwerbrand, 2008). The comprehensiveness of the proposals differs, but on a general level they are similar with only minor variations. Different aspects of larger scale beams have been studied by e.g. Noghabai (2000), Özcan *et al.* (2009), Slater *et al.* (2012), Padmarajaiah and Ramaswamy (2002), Altun *et al.* (2007) and Jansson *et al.* (2010). However, there is a lack of studies covering both numerical modelling and analytical design calculations of structural components relevant to practical applications. A study addressing this is presented in Paper B, where three concrete beams subjected to four-point bending were studied through experiments, analyses in accordance with Model Code 2010, and finite element analyses. The purpose of this study was to investigate the applicability of Model Code 2010 and the non-linear finite element method in designing of fibre reinforced concrete structures. The experimental set-up for the beams is shown in Figure 5.1, and the three beam configurations are specified in Table 5.1.

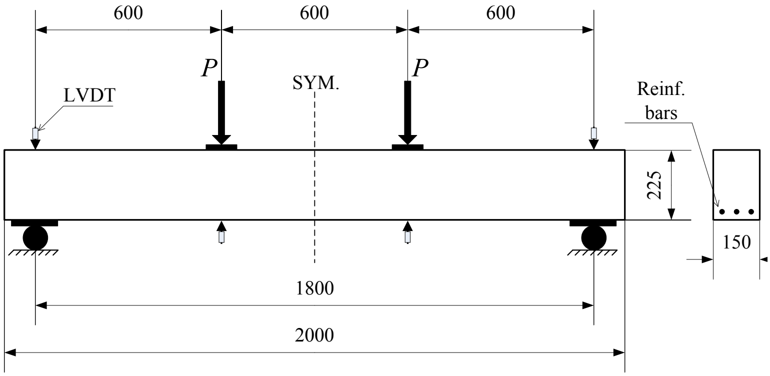


Figure 5.1: *Experimental set-up.*

Table 5.1: Test beam configurations.

Beam No.	I	II	III
V_f, nominal	–	0.25%	0.50%
V_f, actual (mean value from wash-out)	–	0.18%	0.45%
Reinforcement	$3\phi8$	$3\phi6$	$3\phi8$
$f_\mathrm{ccm.28d}$	58.8 MPa	58.1 MPa	57.5 MPa
$f_\mathrm{ctm.28d}$	2.9 MPa	2.7 MPa	3.0–3.1 MPa
E_cm	32.5 GPa	30.5 GPa	31.0 GPa

A key feature of this study was that all material parameters of the fibre reinforced concrete were determined based on an extensive experimental programme. Other parts of the programme are given in Jansson *et al.* (2012a) and Jansson *et al.* (2012b), describing the experiments that provide material properties for concrete in tension, bond behaviour and reinforcement steel. These experimental results, somewhat idealised, were used as input for the finite element analysis.

5.1 Finite element analysis

The finite element analyses were carried out using a two-dimensional plane stress model. Four-node quadrilateral isoperimetric elements arranged in a dense quadratic mesh (element size: 5mm) were used. The reinforcement was modelled by truss elements connected to two-dimensional interface elements providing the bond-slip properties. Furthermore, a smeared crack approach using rotating cracks was used. The deformation controlled analysis was devised in two phases: first, the selfweight was applied, followed by incremental loading in phase two. Supports were modelled using eccentric tying, i.e. nodes representing the support plate were maintained on a straight line intersecting the plate center node. More details are found in Paper B.

Moreover, finite element analyses were also made with modified bond stress versus slip relation, according to Engström (1992), to include the effects of yielding reinforcement. It is reasonable to assume that the bond would decrease once the reinforcement started to yield (Figure 5.2). As expected, this assumption led to more localised cracks, as no new cracks formed once yielding had occurred.

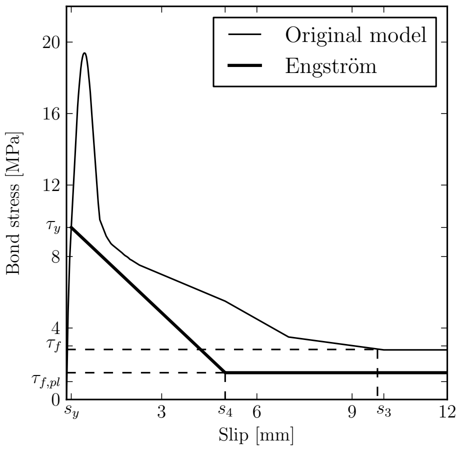


Figure 5.2: *Bond versus stress relation modified according to Engström (1992), here exemplified for the beam with* $V_{\mathrm{f}} = 0.5\%$.

5.2 Model Code 2010

Model Code 2010 (MC 2010) presents an analytical approach to the design of steel fibre reinforced concrete structures; and is a joint effort of leading researchers. The suggestion applied in this section had in many respects been adopted to MC 2010 from di Prisco *et al.* (2009) and is further described in di Prisco *et al.* 2013. To facilitate design calculation of the beams in this study, the residual flexural strength of the steel fibre reinforced concrete was derived using a non-linear finite element model, in a similar fashion as presented in Section 4.3; the results from uni-axial testing were used as input to a model of a three-point bending beam. In Figure 5.3, the result from the modelled three-point bending beam with $V_{\mathrm{f}} = 0.25$ % is presented.

The residual flexural strengths, $f_{\mathrm{R},j}$, at the crack mouth opening displacements $CMOD_1 = 0.5\,\mathrm{mm}$, $CMOD_3 = 2.5\,\mathrm{mm}$ and $CMOD_4 = 3.5\,\mathrm{mm}$, was calculated in accordance with MC 2010 as

$$f_{\mathrm{R},j} = \frac{3F_j l}{2bh_{\mathrm{sp}}^2}, \tag{5.1}$$

where F_j is the load applied at $CMOD_j$, l is the span length of the beam, b is the depth of the beam, and h_{sp} is the distance between the notch tip and the top of the specimen. In Table 5.2, the residual flexural strengths, $f_{\mathrm{R},j}$ are presented for the different material compositions.

In compression, MC 2010 refers to the response of plain concrete. The stress in the

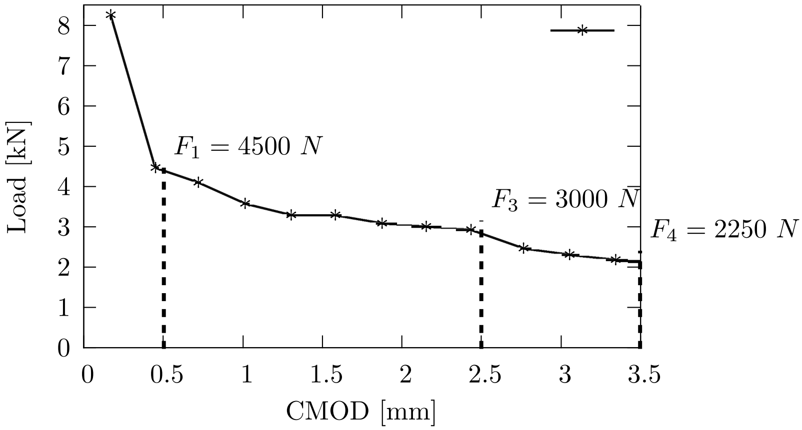


Figure 5.3: *Load versus crack mouth opening displacement (CMOD) in finite element analysis of three point bending test with* $V_\mathrm{f} = 0.25\%$.

Table 5.2: The residual flexural strengths presented together with the related load of the load-CMOD curve.

V_f	F_1 [N]	F_3 [N]	F_4 [N]	f_R1 [MPa]	f_R3 [MPa]	f_R4 [MPa]
0%	0	0	0	0	0	0
0.25%	4500	3000	2250	1.44	0.96	0.72
0.50%	8024	7381	5355	2.57	2.36	1.71

compressive zone in the ultimate limit state is assumed to equal the compressive strength and act on the compressive height reduced with a scalar, $\lambda = 0.8$ (Table 5.3).

In tension, MC 2010 provides the opportunity to choose from two simplified stress-crack opening constitutive laws: a plastic rigid response and a linear softening response. The stresses that represent these constitutive laws are deduced from the residual flexural strength, $f_{\mathrm{R},j}$. In this study, the linear post-cracking model was adopted calculating the sectional moment resistance at yielding of the reinforcement and in the ultimate stage a plastic stress distribution was assumed. The models are based on the reference stresses f_Fts and f_Ftu that represent the residual strength for serviceability crack openings and ultimate residual strength, respectively. According to MC 2010, the reference stresses are calculated as

$$f_\mathrm{Ftsm} = 0.45 f_\mathrm{R1}, \tag{5.2}$$

and

$$f_\mathrm{Ftu} = f_\mathrm{Ftsm} - \frac{w_\mathrm{u}}{CMOD_3}(f_\mathrm{Ftsm} - 0.5 f_\mathrm{R3} + 0.2 f_\mathrm{R1}), \tag{5.3}$$

or

$$f_{\mathrm{Ftu}} = \frac{f_{\mathrm{R3}}}{3} = 0.915\mathrm{MPa}, \tag{5.4}$$

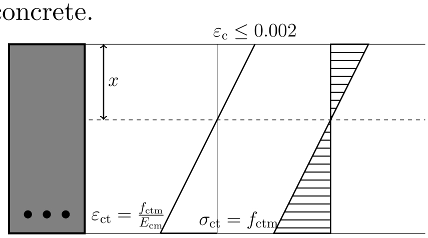
depending on the stress distribution, where Equations 5.2 and 5.3 describe the reference stress for a linear distribution, and Equation 5.4 is valid for a plastic distribution. In Table 5.3, the stress distributions over a cross-section are exemplified, for the yielding and at failure, respectively. In order to calculate f_{Ftu} of the linear distribution, the permitted crack width, w_{u}, must be defined. MC 2010 proposes using the permitted crack width as design criterion; however, if not defined, w_{u} can be assumed to equal $CMOD_3$, which was done in this study, i.e. $w_{\mathrm{u}} = CMOD_3 = 2.5$ mm. The ultimate tensile strain ($\varepsilon_{\mathrm{fu}}$) was, considering the experimentally obtained crack spacing, slightly conservatively chosen at 2%. According to MC 2010 the ultimate tensile strain can be chosen as a function of either the tensile zone height or the crack spacing, derived from a characteristic length theory.

In MC 2010, three conditions are given for the ultimate sectional moment capacity; one of these three conditions is limiting. In practice, one of them, the attainment of the ultimate tensile strain of the conventional reinforcement is not relevant in most cases, as it is mainly of concern when cold-drawn reinforcement bars are used. The remaining two conditions treat the attainment of the maximum tensile strain ($\varepsilon_{\mathrm{fu}}$) or maximum compressive strain ($\varepsilon_{\mathrm{cu}}$), respectively. As formulated in MC 2010, it was not evident for the author of this thesis which criteria should be chosen in design: either the failure criteria obtained at the lowest curvature, or the criteria resulting in the lowest, or even possibly highest, ultimate load. Adopting the cross-sectional model resulting in the lowest curvature would provide the solution that physically occurs first; however, the cross-sectional model resulting in the lowest moment capacity (or load) would always reflect the most conservative option. Furthermore, the application of the attainment of the ultimate tensile strain as a failure criterion could be discussed. The tensile behaviour of steel fibre reinforced concrete is ductile by nature; this is the greatest asset of the material. Thus, considering the attainment of the ultimate tensile strain, in only one part of the cross-section, as a failure, could be limiting; depending on the cross-section and material parameters, higher moment capacity could be obtained for increasing curvatures until the compressive ultimate strain is reached.

Table 5.3: The stress distributions used for calculating the response at: cracking, yielding of conventional reinforcement, and ultimate limit state.

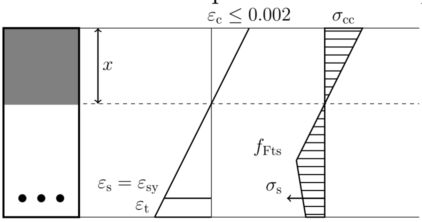
Crack initiation

At this point, the section is uncracked and the response of compressed concrete is linear. The sectional response is determined by the maximum tensile strength of the concrete.



Yielding of reinforcement

The response of compressed concrete is linear and the reinforcement is starting to yield. The maximum tensile strain, ε_t, in the cracked concrete has not reached the ultimate tensile strain; thus, the softening curve is defined by Equation 5.2 and Equation 5.3. To calculate the contribution, the stress level at ε_t needs to be calculated from the linear relation of Equation 5.2 and Equation 5.3.

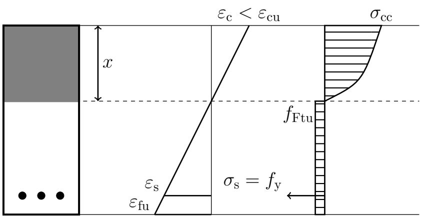


Failure in the FRC by reaching the maximum strain in

tension **or** **compression**

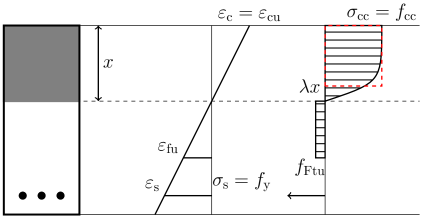
tension

The response of compressed concrete is non-linear but has not reached the ultimate compressive stress block; hence, a iterative procedure is needed. The reinforcement is yielding and the ultimate tensile strain of the fibre reinforced concrete is reached at the bottom of the section.



compression

The response of compressed concrete is non-linear and the reinforcement is yielding. If the maximum tensile strain is larger than the ultimate tensile strain in the fibre reinforced concrete, the contribution of fibre reinforced concrete in tension is limited to the part of the section where $\varepsilon_t \leq \varepsilon_{fu}$.



5.3 Results and discussion

Analytical results obtained using both sectional models, as discussed in previous section, are presented in Table 5.4. The reinforcement was assumed to be elastic ideal plastic. The tensile failure, as defined in MC 2010, occurred at a considerably lower curvature, but with a higher moment capacity than the attainment of the compressive strain. Thus, according to these analytical results, after the tensile strain was reached, the beam section exhibited a softening behaviour until the compressive failure was reached.

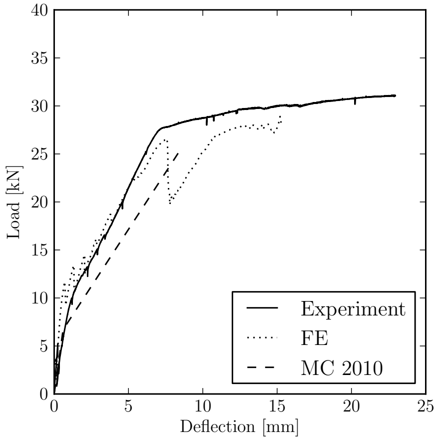
Table 5.4: Obtained cross-sectional moment capacity (M_{Rd}), curvature ($1/r$), and corresponding ultimate load (P_{u}), under the different assumptions for the ultimate cross-section as presented in Table 5.3.

Failure assumption	V_{f}=0.25% M_{Rd} [kN m] (P_{u} [kN])	V_{f}=0.25% $1/r$ [1/m]	V_{f}=0.50% M_{Rd} [kN m] (P_{u} [kN])	V_{f}=0.50% $1/r$ [1/m]
Tensile strain	12.8 (21.3)	0.095	19.4 (32.3)	0.097
Compressive strain	11.9 (19.8)	0.395	17.2 (28.7)	0.254

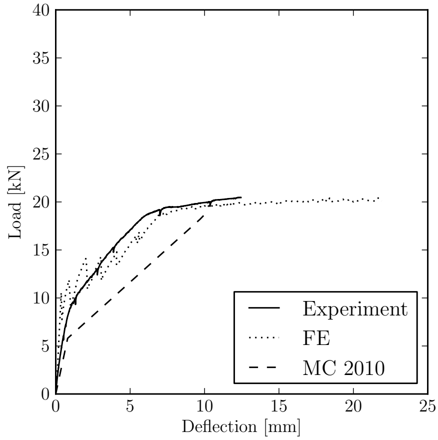
To further describe the flexural response of the reinforced beams, three points in the load-deflection curve were calculated: crack initiation, yielding of reinforcement, and at failure, see Figure 5.4. In addition, the corresponding deflections were calculated by using the measured value of the Young's modulus and the moment of inertia of the equivalent concrete cross-section. It can be argued that the cracked fibre reinforced concrete might contribute to the equivalent concrete section. However, since MC 2010 does not include this effect, it was left out of this study. The resulting loads and deflections are presented in Figure 5.4.

Good agreement was found when comparing experiments with numerical analyses, see Figure 5.4, where both experimental, analytical and numerical results, in terms of load-deflection behaviour, are presented. In these results, only the attainment of the compressive strain is shown as a failure criterion; hence, the additional cross-sectional state where the tensile strain is reached is not included in the figure. The load-carrying capacity and the stiffness in the cracked stage from Model Code 2010 were underestimated, assuming a compressive failure. Choosing the, in this case, less conservative failure mode of reaching the ultimate tensile strain, a more accurate result would be obtained, see Table 5.4.

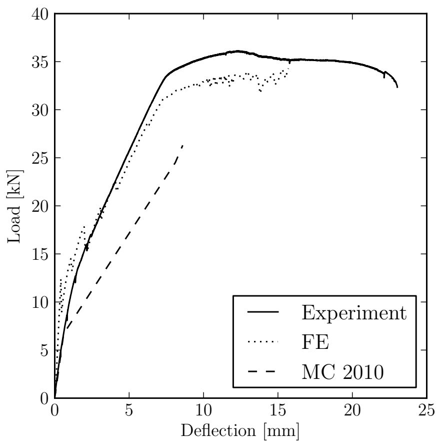
Crack patterns in experiments and from finite element analyses are compared in Figure 5.5. The number of cracks, the total spread and crack distance roughly agree. Differences can be attributed to imperfections, e.g in the sample and set-up. In Figure 5.5, half of the beams tested are displayed in order to facilitate the comparison. The omitted halves of the real beams had similar, but not exactly matching, patterns. Furthermore, both in tests and numerical analyses the number of cracks tends to increase with increasing fibre content. This relation was expected since it has been previously observed by e.g. Bischoff (2003), Jansson *et al.* (2012b) and Lawler *et al.* (2005).



(a) V_f = *0.00%, 3ϕ8*

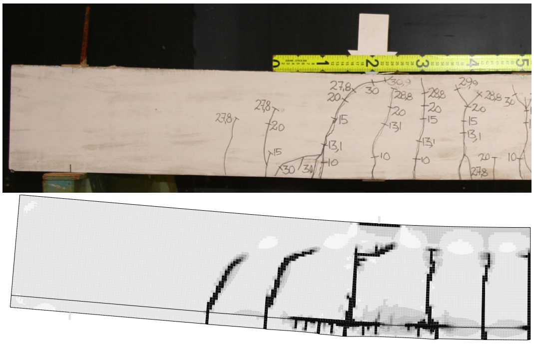


(b) V_f = *0.25%, 3ϕ6*

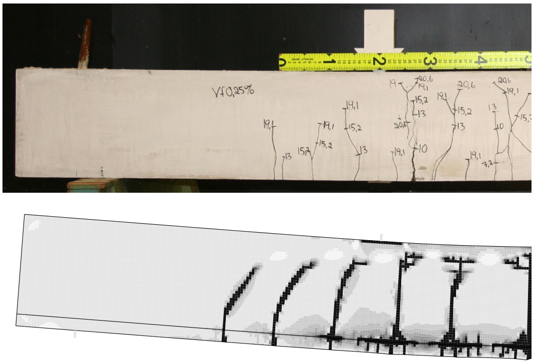


(c) V_f = *0.50%, 3ϕ8*

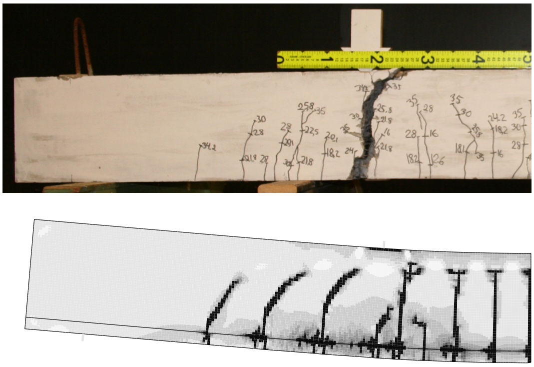
Figure 5.4: *Load-deflection (mid-span) behaviour: comparison between experiments, analysis according to MC 2010 and FE modelling utilizing bond model according to Engström (1992).*



(a) *Beam I, V_f = 0.00%, 3ϕ8*



(b) *Beam II, V_f = 0.25%, 3ϕ6*



(c) *Beam III, V_f = 0.50%, 3ϕ8*

Figure 5.5: *Crack patterns in experiments compared with those from FE analyses (using improved bond model). The deformation applied (in FE analyses) was 5.5 mm, 6.4 mm and 13.0 mm for V_f = 0%, V_f = 0.25% and V_f = 0.50%, respectively.*

To summarise, good agreement was obtained using numerical methods. Using the analytical method proposed in MC 2010, the load carrying capacity was underestimated; however, the method was applied under conservative assumptions. Furthermore, this study was only conducted on one specific type of structural member. The assessment of load-carrying capacity of steel fibre reinforced structures, both through design methods and numerical analyses, is subject to future research. In Chapter 6, a statically indeterminate structure is studied as an extension of the research presented.

6 Reinforced concrete slabs

The investigation of reinforcement alternatives for tailor-made concrete structures (Chapter 2) concluded in several topics for further research with regards to the chosen alternatives. To address these, an experimental programme was initiated, focusing on the following research questions:

- How do steel fibres affect the load redistribution in statically indeterminate structures? Redistribution is an important prerequisite in many design methods.
- How is a load distributed along the length of a support? Do steel fibres affect this distribution?
- Does the MC 2010 code proposal appropriately assess the load carrying capacity added by steel fibres in statically indeterminate structures? Is non-linear finite element analysis a suitable method for assessing the structural behaviour of such structures?

The intention was to develop a test set-up, in which the structural behaviour of reinforced concrete structures with steel fibre reinforcement could be assessed with regard to design calculation as well as numerical analysis, both alone and in combination with conventional reinforcement. Moreover, to further study the influence of steel fibres on the load distribution would be desirable. The experimental programme is described in detail in Paper C, including the development of the test set-up, results and an overview of experimental configurations from the literature.

The test set-up, which is described in the following section, originated in the idea of testing a part of a complex concrete structures which in the context of this research project was assumed to resemble a shell-like structure. However, due to the difficulties associated with testing a shell-like structure, the test set-up was simplified to a flat slab, thereby keeping one of the most significant structural features, the statical indeterminacy. To avoid diffuse crack patterns, the corners of the square slab were left out, resulting in octagonal test specimens. The test set-up is presented in Figure 6.1.

Even though the span lengths were equal, the specimens containing conventional reinforcement bars were designed with uneven reinforcement distribution, in order to enforce load redistribution in the non-linear stages. The reinforcement was cast with a concrete cover of 20 and 26 mm in the strong and weak direction, respectively. All slab configurations tested are presented in Table 6.1. In addition to these slabs, five similar specimens with textile reinforcement were tested (Williams Portal *et al.* 2013).

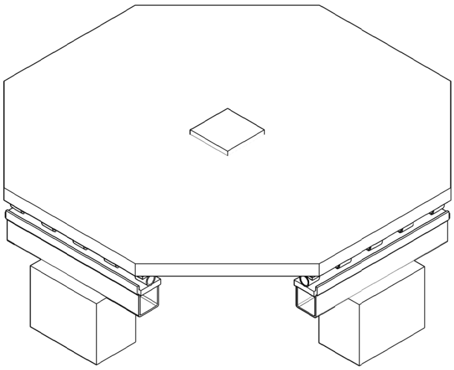


Figure 6.1: *Test set-up.*

Table 6.1: Slab configurations. CR denotes slabs with conventional reinforcement bars as only reinforcement type. CFR denotes slabs with both conventional reinforcement bars and steel fibre reinforcement. FR denotes slabs with steel fibre reinforcement alone.

Type	#	Reinforcement bars	Steel fibres
CR	3	ϕ6 (s194/96 mm) B500C	-
CFR	3	ϕ6 (s194/96 mm) B500C	35 kg/m^3 (DRAMIX 5D)
FR	3	-	35 kg/m^3 (DRAMIX 5D)

6.1 Test set-up

The 2.4 m wide octagonal slabs were supported on 20 high-tolerance steel pipes and loaded with a point load applied to a loading plate in the centre (Figure 6.1). To facilitate monitoring of the load distribution, the steel pipes were instrumented with two strain gauges each. The roller size was chosen so that the strains under the expected load range were measurable, without exceeding the yield strain.

All slabs and specimens for determination of the material properties simultaneously cast with concrete delivered from a ready-mix plant. Two batches of concrete were delivered, one with plain concrete and another with steel fibre reinforced concrete. Both mix compositions can be found in Papers C and D, where the material testing is further described. The average material properties are provided in Table 6.2.

Table 6.2: Concrete mix compositions and properties.

	Plain concrete	Fibre reinforced concrete
$f_{\mathrm{c,mean}}$ [MPa]	50.9	44.3
E_{c} [GPa]	31.7	30.9
$f_{\mathrm{ct,mean}}$ [MPa]	2.70	2.99

6.2 Results

Comparing the test results from all nine slabs, as presented in Figure 6.2, the influence of steel fibres is evident. All slabs with conventional reinforcement (CR) showed similar behaviour; elastic until cracking at 25-30 kN followed by a clear bending hardening behaviour. The slabs with combined reinforcement (CFR) acted similarly, except that they exhibited greater stiffness during the cracked hardening stage in addition to a higher load-carrying capacity. The slabs reinforced with steel fibres alone (FR) showed no bending hardening, i.e. the cracking load was the highest load applied during these tests. Results are considered to be consistent, with the exception for one of the slabs with combined reinforcement (CFR1). This slab was unintentionally cast approximately 10% thicker than the other slabs, corresponding to the higher load-carrying capacity. It is worth pointing out that the scatter observed in uni-axial tensile tests, as well as the three-point bending tests (Chapter 4), is not seen in the results of the slab tests. This indicates that steel fibre reinforced concrete might be considered a homogeneous material when approaching a structural scale. This effect is believed to be attributed to two reasons. Firstly, material testing was conducted on notched specimens. Fracture consequently occurred in the notched section; hence, the fibre content of that section heavily affects the material behaviour, leading to high scatter. Secondly, as the scale of the structural system increases, the redundancy for local weaknesses increases. The cracks need to propagate further in the slabs tested than in the smaller specimens. During the crack propagation, stronger and weaker regions will be intersected; however, the average characteristics will be governing. Furthermore, the structural redundancy obtained from having two load-carrying directions might have an averaging effect.

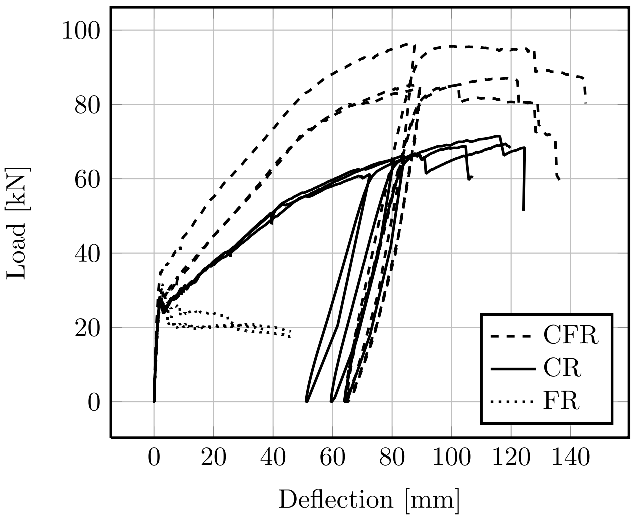


Figure 6.2: *Load versus mid-deflection, for all tested slabs.*

In specimens without conventional reinforcement, no additional major cracks were formed once the ultimate crack pattern had been obtained; this occurred almost immediately following the first cracking, agreeing well with the non-bending hardening behaviour showed in Figure 6.2. The final crack pattern of slab FR1 is shown in Figure 6.3.

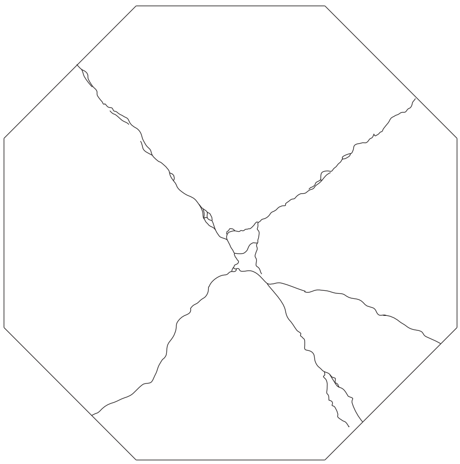


Figure 6.3: *Crack pattern after slab tested, with fibre reinforcement alone (Specimen FR1).*

Figures 6.4 and 6.5 show sketches of the final crack patterns for the slab types with conventional reinforcement. Generally, initial cracks ranged from the centre of the slab diagonally towards the unsupported edges. Considering the number of cracks obtained, a difference emerged between the slabs including conventional reinforcement and those with steel fibres alone. Furthermore, it was observed that the number of cracks was higher in the slabs with both conventional and steel fibre reinforcement than in the slabs with conventional reinforcement alone. Accordingly, the crack widths were considerably reduced by adding steel fibres compared to the slabs with conventional reinforcement alone. This observation agrees well with the previously known behaviour of steel fibre reinforced concrete; the number of cracks increases, while the crack width is generally smaller (e.g. Vandewalle, 2000 or Bischoff, 2003). Although only one slab of each reinforcement configuration is presented here, the remainder of the series showed similar behaviour.

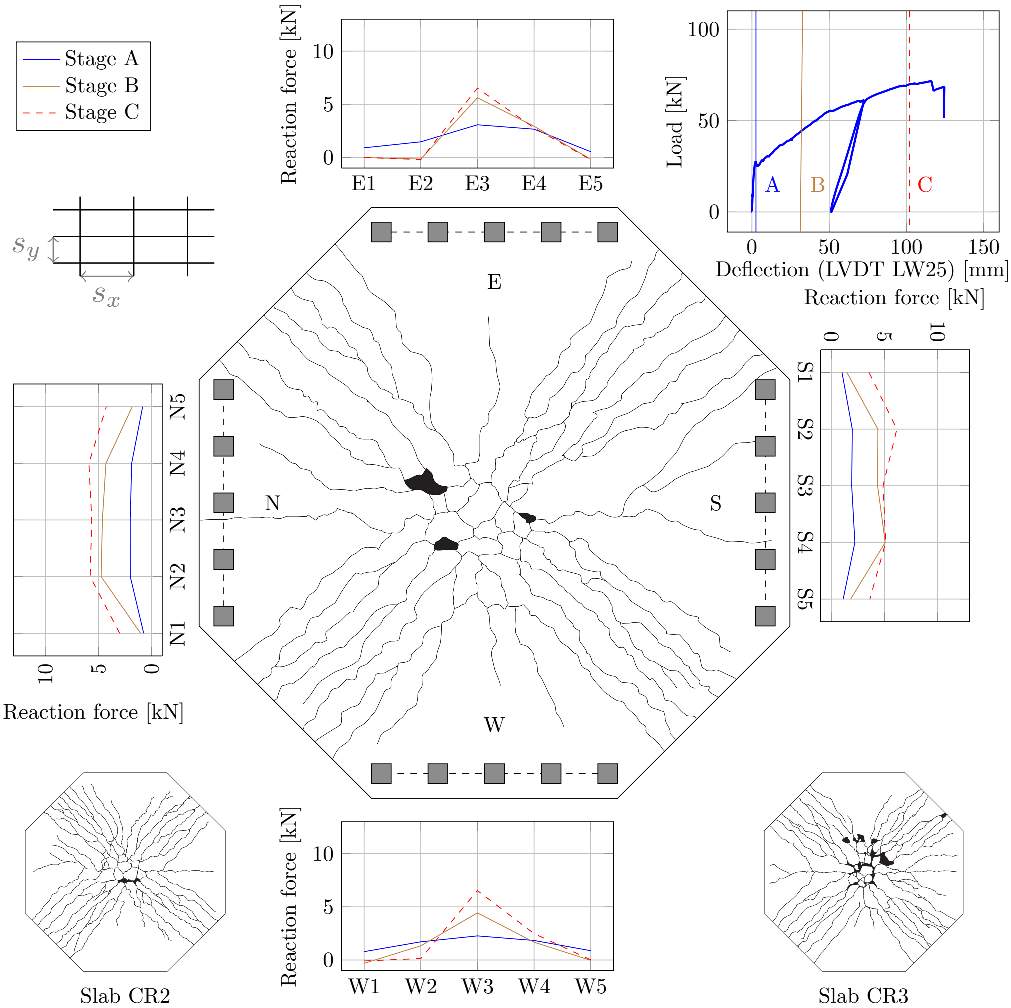


Figure 6.4: *Crack patterns and reaction force distribution over the support length in a specimen with conventional reinforcement alone (CR1). The reaction forces are provided for three stages (A-C) as defined in the load versus displacement plot in the top right corner.*

The load distribution along the supported edges is visualised in Figures 6.4 and 6.5. The reaction force prior to loading (i.e. the self-weight) has been deducted from all reaction forces presented. Comparing the reaction forces measured through the strains in the supporting rollers with the applied load (see Paper C), it can be concluded that reasonable agreement was obtained; hence, the approach was successful. It was observed that for both CR and CFR slab types, the reaction forces over the supports were more equally distributed in the strong than in the weak direction. Thus, the denser reinforcement perpendicular to the support transfers the load transversely to a higher extent, leading to

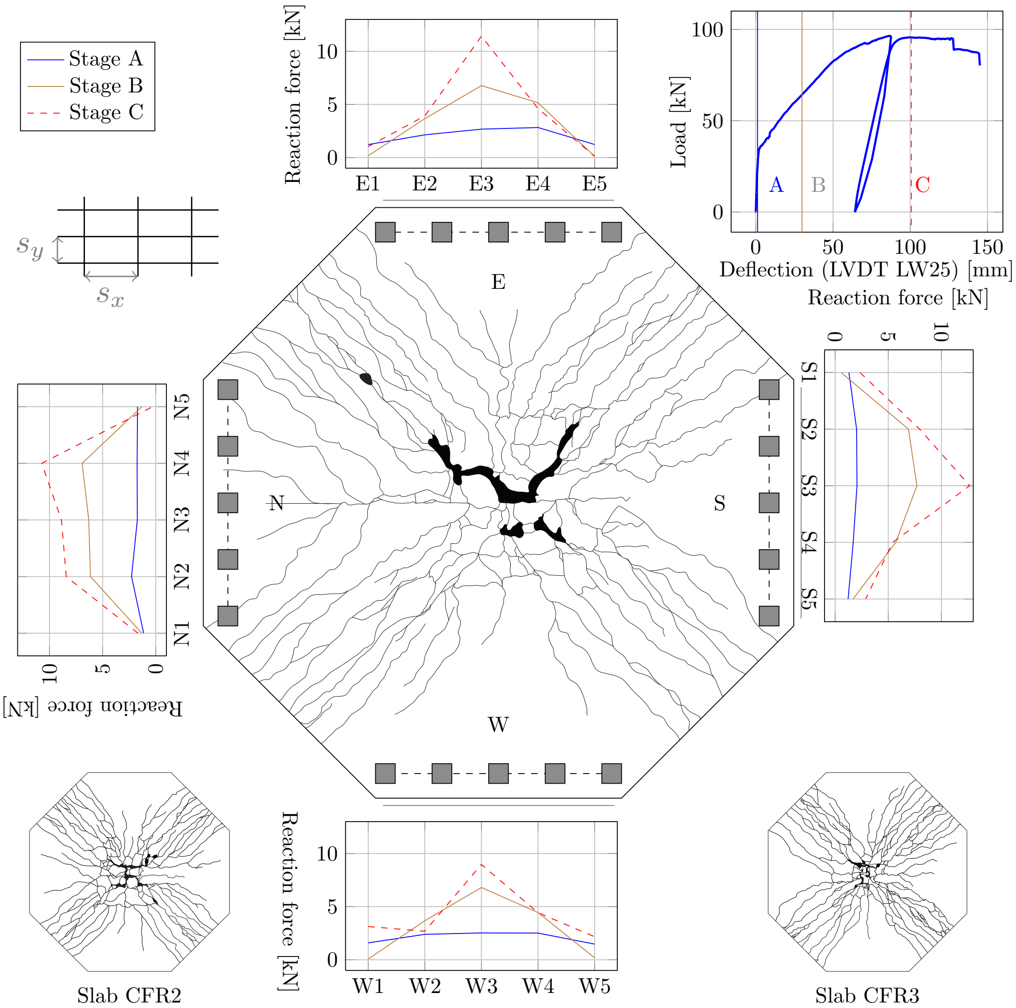
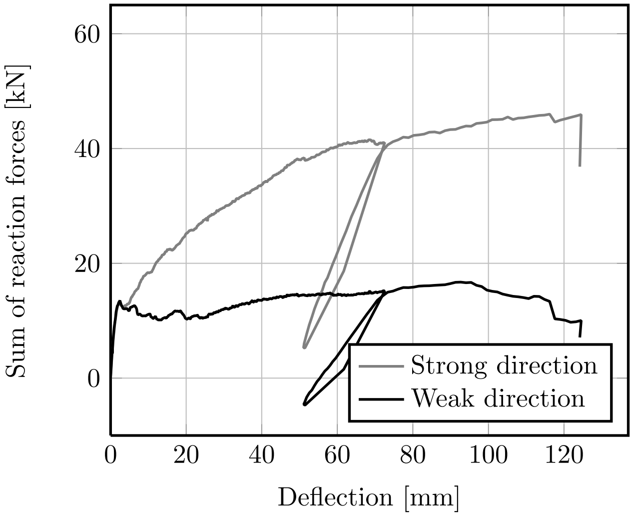


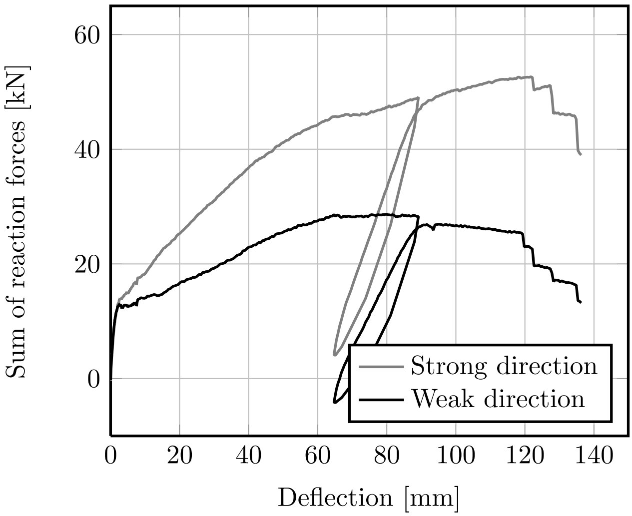
Figure 6.5: *Crack patterns and reaction force distribution over the support length in a specimen with both conventional and steel fibre reinforcement (CFR1). The reaction forces are provided for three stages (A-C) as defined in the load versus displacement plot in the top right corner.*

more evenly distributed reaction forces. The fibres caused a slightly more even distribution along the support lines in the weak direction, especially at large deflections (Stage C, ≈100 mm), where larger reaction forces were obtained in the outer rollers in the weak direction as well. In fact, it was observed that only a part of the support provided in the weak direction is used in the slabs with conventional reinforcement alone. In the slabs with both reinforcement types, the difference in terms of utilized support length, observed between the strong and weak direction, is evened out. This trend is further quantified in Paper C.

The sum of the reaction forces at the supports in the strong and weak directions are presented in Figure 6.6. For clarity, only results from two selected slabs are presented. Similar behaviour was, however, observed throughout the test series, as seen in Figure 6.7, where the support reactions in relation to the total load are presented for all slabs. The load redistribution taking place as cracking occurs depended on the differing conventional reinforcement content. In addition, comparing the reaction force distribution between the strong and weak directions, the strong influence of steel fibres was observed. In the CFR series, the load carried by the supports in the weak direction continued to increase after cracking, while only a minor increase of the support reaction in the weak direction was observed in the CR series, corresponding to strain hardening of the reinforcement. It could be claimed that the main part of the additional load carrying capacity was obtained by providing better redistribution capacity through the addition of steel fibres. The load carried by the supports in the weak direction increased by as much as about 60% through the presence of steel fibres. On the other hand, the load carried by the supports in the strong direction only increased by about 13% through the presence of fibres. In absolute numbers, the increase in the weak direction is larger than in the strong direction, 25 kN compared to 6 kN.

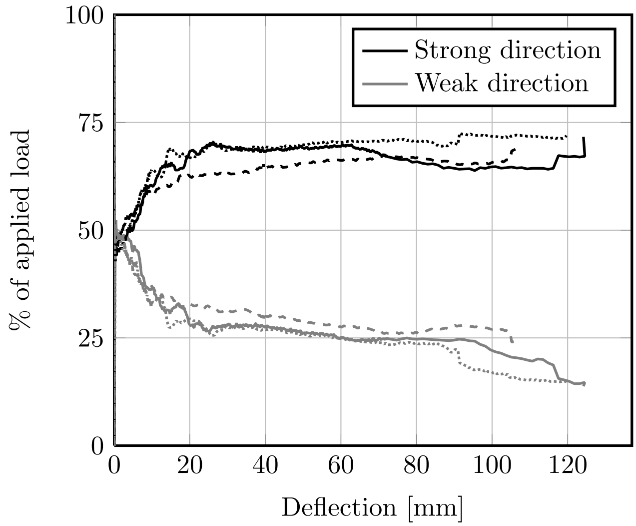


(a) *Conventional reinforcement alone (CR1)*

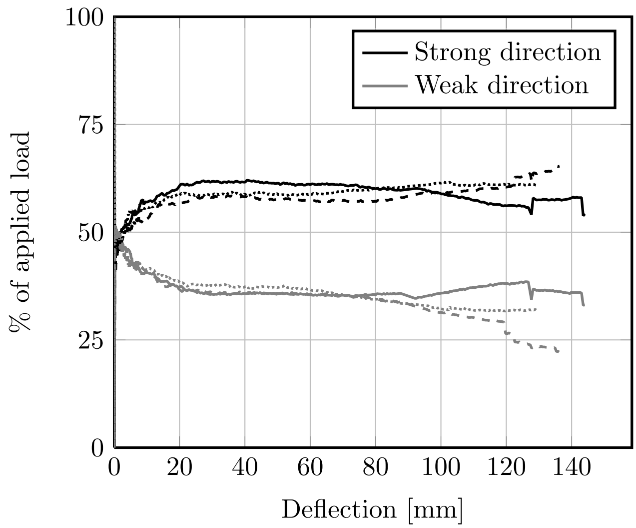


(b) *Conventional reinforcement and steel fibre reinforcement (CFR2)*

Figure 6.6: *Total reaction force per supported edge.*



(a) *Conventional reinforcement only (CR)*



(b) *Conventional and steel fibre reinforcement (CFR)*

Figure 6.7: *Ratio of load carried by the supports in the strong and weak direction, respectively, for CR slabs (a) and CFR slabs (b). In both series, the line type denotes the specimen number (1-solid ,2-dashed, 3-dotted).*

6.3 Analytical assessment of the load-carrying capacity

The ultimate load of the slabs was assessed using the yield line method. Furthermore, the design was verified by a three-dimensional finite element model using estimated material properties.

The yield line method is a well-recognised plastic analysis method relying on the choice of failure mechanism (Johansen, 1972). It is an upper bound approach; hence, there might be a more critical failure mechanism forming at a load lower than the one calculated. Using yield line analysis in experimental work, the failure mechanism assumed can be verified against the actual failure mode, assuming that the crack pattern conforms to the yield lines. In the current study, the assumed failure mode matched the initial crack pattern and the major cracks at the ultimate stage.

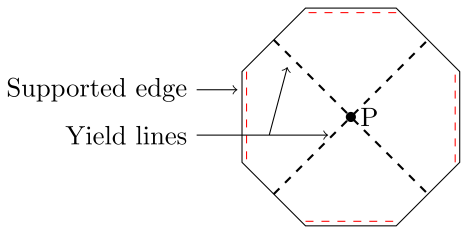


Figure 6.8: *Assumed failure mode.*

Assuming the mechanism in Figure 6.8 and using the cross-sectional capacity obtained from a detailed sectional analysis, the failure load (using conventionally reinforced concrete alone) was calculated as $P = 39.4$ kN. The average corresponding ultimate load obtained from experiments was 69.9 kN. The underestimated capacity may seem surprising as the yield line solution provides an upper bound; however, this underestimation is mainly attributed to two effects not included in the analytical method, the lack of strain hardening in the yield line model and membrane effects:

- Using the yield line method, the reinforcement was considered ideal plastic; hence, the tension hardening observed for the reinforcement bars was neglected. The contrary approach would be to instead calculate the moment capacity using the ultimate strength of the reinforcement (666 MPa, per Paper C). Using the same yield line solution, this approach would increase the estimated ultimate load to 49.0 kN.

- Membrane forces predominantly affect horizontally restrained structures. However, due to the multi-directional load carrying, membrane effects occur at large deflections in slabs without horizontal restraint as documented by Ockleston (1955). In brief, as tensile and compressive in-plane stresses arise, the load capacity increases. Considering the deflection of a one-way slab, as the slab deforms, the edges move towards the centre. If this movement is prevented, tensile stresses will be created in the slab. In a two-way slab, simply supported on four edges, the perpendicular load-carrying direction restrains the horizontal movement of the other direction, giving rise to in-plane forces similarly to a horizontally restrained one-way slab. Furthermore, tensile stresses will arise in the counteracting direction. Consequently, in a two-way slab, these stresses will occur in both directions, resulting in tensile in-plane forces in the centre of the slab surrounded by a ring of compressive forces. The yield moment would increase in the areas with compressive forces. The results from Bailey (2001) indicate that the increasing factor for the ultimate load of a simply supported square slab with the displacement/effective depth-ratio of the slabs in this study would be approximately 1.5. This may explain the major difference between the yield line analysis and the experimental capacity of the slabs reinforced with conventional reinforcement alone.

To include the effect of steel fibre reinforcement in the analytical solution, the sectional moment resistance was calculated utilising the stress and strain distribution suggested in Model Code 2010 (MC 2010). The sectional moment capacity was obtained using the ultimate sectional models, as discussed in Section 5.2. Material properties were evaluated from the three-point bending tests (Section 4.2), resulting in an average rigid-plastic ultimate residual strength, f_{Ftu} =0.92 MPa. The corresponding ultimate tensile strain was chosen at $\varepsilon_{\mathrm{fu}} = 2\%$. Considerably higher capacities were obtained considering the attainment of the ultimate tensile strain of the fibre reinforced concrete as the governing failure criteria instead of assuming a compressive failure; the ultimate tensile strain was reached at lower curvature. Hence, it could be argued that this capacity is correct to use, cf. discussions in Section 5.2. Using the yield line method, as presented in Paper C, the ultimate load carrying capacities obtained were 55.5 kN and 41.1 kN for both sectional models, respectively. These findings may be compared with the capacity calculated for a slab with conventional reinforcement alone: 39.4 kN; hence the estimated contribution of the steel fibres to the ultimate load carrying capacity was about 4% or 40% depending on the assumption made for the cross-section. The absolute capacity increase considering the less conservative assumption (14.4 kN) corresponds roughly to the experimentally obtained absolute increase between the CR and CFR-series. It should be mentioned that the concrete properties were slightly different when comparing plain and fibre reinforced concrete (Table 6.2). The analytical solutions are compared to the experimentally obtained ultimate loads in Figure 6.9, where the consistent underestimation of the different yield line solutions is evident.

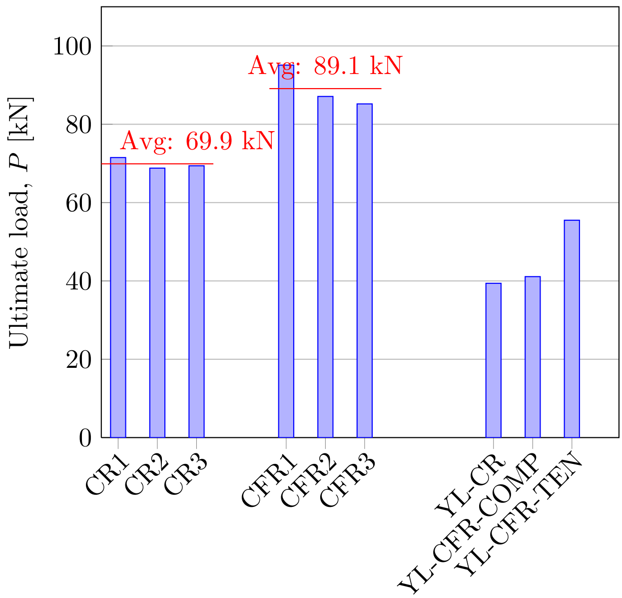


Figure 6.9: *Comparison of experimentally and analytically obtained ultimate loads. YL-CR denotes yield line analysis results obtained for plain concrete, using a detailed sectional analysis. YL-CFR-COMP is the result obtained by adopting the ultimate compressive strain as the governing failure mode. By instead using the attainment of ultimate tensile strain as the definition of failure, the result denoted YL-CFR-TEN was obtained.*

6.4 Numerical approach

As just shown, the analytical models underestimated the load-carrying capacity of the slabs. Refined numerical methods, e.g. non-linear finite element analyses, can provide a more precise assessment; this was investigated in the following studies:

- In Irani and Mazhari Abadi (2013), a model based on **shell elements** and fully bonded reinforcement was used (Figure 6.10). Each element was constituted by three in-plane integration points and eleven integration points over the thickness. Each support was modelled by restricting downward movement of three nodes per roller, while upward movement was allowed. Practically, this boundary condition was applied using an interface element between the slab and a vertical support. The interface element was assigned a linear elastic fictitious material, extremely stiff in compression but very weak in tension. The crack band width was chosen to the element size (35 mm).

- Jiangpeng *et al.* (2014) present an approach using **solid elements** (Figure 6.11). The conventional reinforcement was modelled using two approaches: embedded with full interaction and embedded with a given bond-slip relation (based on the Model Code 2010 suggestions). Here, only the results from the model with fully embedded reinforcement are considered; the influence of including the bond behaviour was limited to a more pronounced crack localisation. Steel fibre reinforced concrete was not included in this study. The supports were modelled in detail by including the underlying steel plate. Interface elements were used between the steel plate and the concrete slab, including friction effects. The crack band width was chosen to the average crack spacing (100 mm) and Poisson's ratio $\nu = 0.15$. The analysis was performed both with and without taking non-linear geometrical effects into account.

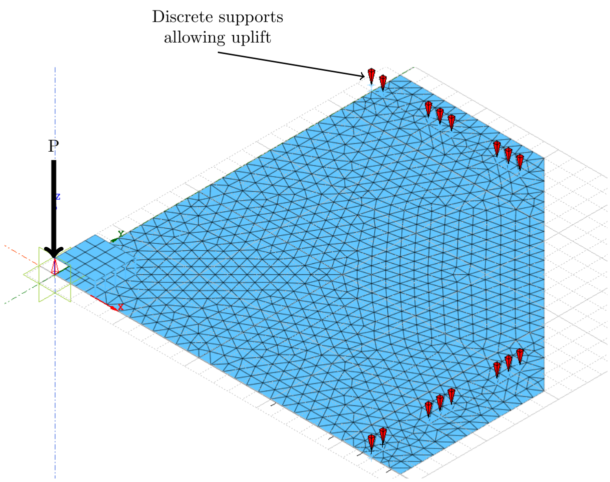


Figure 6.10: *Finite element model of slab, utilizing triangular shell elements of first order. Only a quarter of the slab was modelled, due to symmetry.*

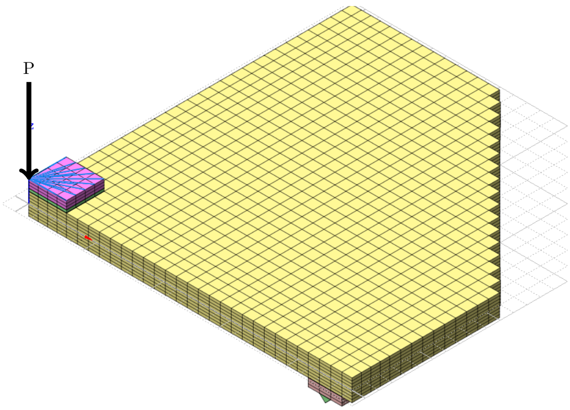
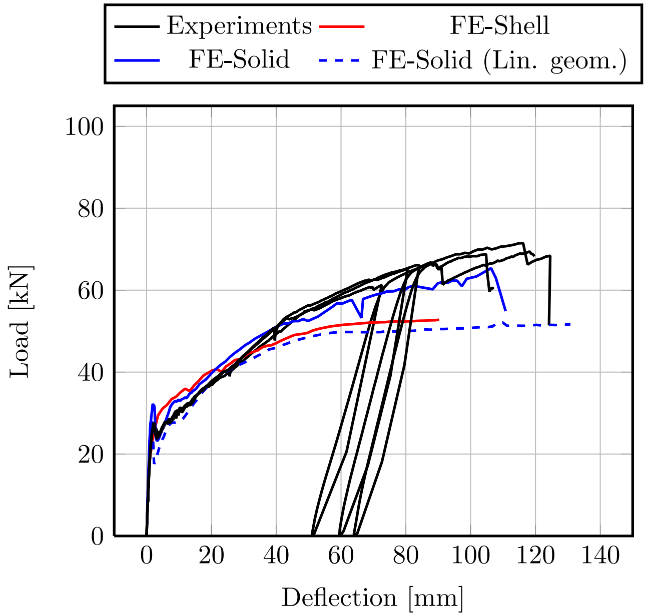


Figure 6.11: *Finite element model of slab, utilizing solid elements. Only a quarter of the slab was modelled, due to symmetry.*

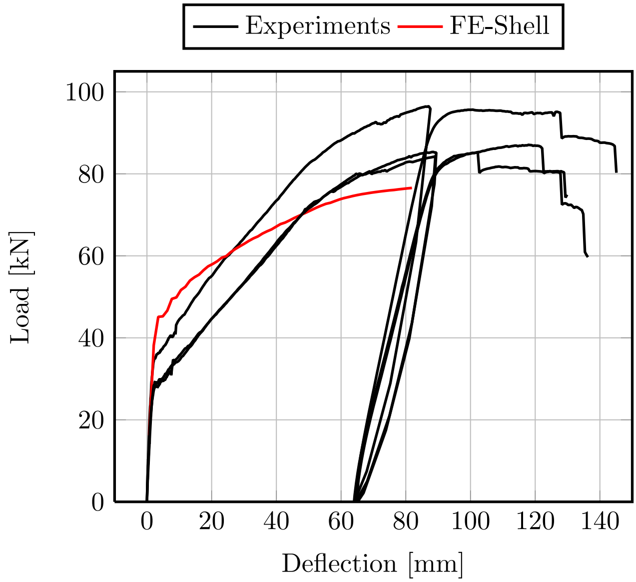
In both studies, material properties were derived from the experimental programme presented in Papers C and D. The results from both finite element analysis approaches are presented in Figure 6.12.

The results show a slight underestimation of the ultimate load in all analyses and both modelling approaches render similar results. The accuracy of the solid model was increased by including non-linear geometrical effects. Thus, by including non-linear geometry, the membrane effects at larger deformations discussed in Section 6.3 could be described (Vecchio and Tang, 1990 and Polak and Vecchio, 1993). Furthermore, it should be noted that for the analyses using shell elements, only the results until the ultimate strain in the reinforcement was reached are shown. The analyses could actually be continued, as the reinforcement after the ultimate strain was assumed to have a plastic response; however, they were considered to not be reliable after this point. To model the deformation capacity, not only the material of the reinforcement needs to be modelled more in detail, also bond between the reinforcement bar and the concrete needs to be included and especially the bond loss at yielding, e.g. as presented in Chapter 5.

In Figure 6.13, the results from the finite element model using shell elements are presented in terms of reaction force per supported edge. Compared to experimental values, it was observed that the behaviour was roughly captured; however, the reaction force increase obtained experimentally in the weak direction was not observed in the analysis results.

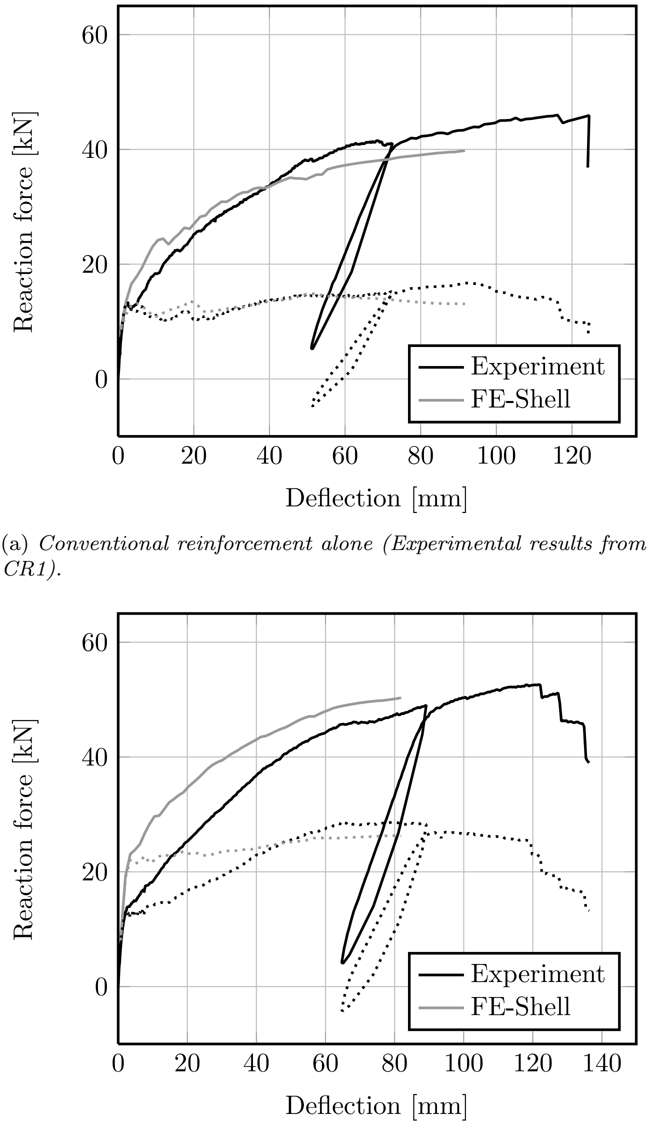


(a) *Conventional reinforcement alone (CR)*



(b) *Conventional and steel fibre reinforcement (CFR)*

Figure 6.12: *Results from the numerical analyses.*



(a) *Conventional reinforcement alone (Experimental results from CR1).*

(b) *Conventional reinforcement and steel fibre reinforcement (Experimental results from CFR2).*

Figure 6.13: *Total reaction force in the strong (solid) and weak (dotted) direction.*

The results of the analyses indicate that a non-linear analysis is suitable for the assessment of steel fibre reinforced concrete structures. It is important to note that the use of advanced numerical tools in a practical context requires higher competence than the use of ordinary design methods to ensure the quality of results. However, used properly by a competent structural engineer, a non-linear analysis provides more than an accurate estimation of the ultimate load in describing the structural behaviour during all stages. Using safety formats with non-linear finite element analysis to account for modelling uncertainties is further discussed in e.g. Schlune (2011).

To summarize the results presented in this chapter, slabs reinforced with both conventional and steel fibre reinforcement exhibited a significantly higher load-carrying capacity than that of the slabs reinforced with conventional reinforcement alone. Furthermore, the steel fibre reinforcement affected the load distribution after cracking. It was also observed that the steel fibres evened out the reaction force distribution along the support length. The analytical model studied underestimated the load-carrying capacity; this is mainly attributed to the lack of strain hardening and membrane effects. By using numerical analyses, the structural behaviour of the slabs were captured; the most accurate results were obtained by including non-linear geometrical effects, as presented by Jiangpeng *et al.* 2014.

7 Conclusions

7.1 General conclusions

An examination of reinforcement alternatives showed that conventional reinforcement is, in most applications, a good solution in terms of the structural integrity provided. However, for some applications, or certain regions within a structure, fibre reinforcement can be beneficial as it could simplify the production process.

The intricacies associated with designing geometrically complex concrete structures and the need for a rational design approach have been discussed in this thesis. Methods for the rational design of conventional reinforced structures, using linear finite element modelling, were exemplified by the implementation of a sandwich analogy. Furthermore, overlooking the entire design process, a digital procedure integrating the different actors would be beneficial.

Considering results from experiments on both beams and slabs, the load-carrying capacity increase provided by the addition of steel fibre reinforcement was evident. In Model Code 2010 (International Federation for Structural Concrete (fib), 2010), a first important step towards a common code has been taken. The models suggested were applied to both beams and slabs, and the result was highly dependent on the assumptions of the cross-sectional level. Interpreting Model Code 2010 conservatively, the load-carrying capacities were underestimated, studying both beams and slabs reinforced with steel fibres as well as conventional reinforcement bars. The slabs tested had a load-carrying capacity considerably higher than the analytically obtained result. This result is, however not mainly a consequence of the model of steel fibre reinforced concrete but also of limitations in the yield line method used, which did not include membrane effects and strain hardening of the conventional reinforcement. From the studies presented in this thesis, it can be concluded that the load-carrying capacity of fibre reinforced beams and slabs could be more accurately assessed by non-linear finite element modelling.

As uniquely shaped structures are likely to be statically indeterminate, the possibility of redistributing the load is necessary in design. The influence of steel fibres was studied in an experimental programme. Testing slabs reinforced with conventional reinforcement alone, as well as with a combination of conventional reinforcement and steel fibre reinforcement, a clear influence of the steel fibres on the load distribution was observed. The conventional reinforcement was asymmetrically arranged so that a weak and a strong direction were obtained. A method to monitor the reaction forces by using strain gauges on steel pipes was developed and successfully utilised. With conventional reinforcement alone, all load increase after the cracking of the slab were transferred to the support in the strong direction. When adding steel fibre reinforcement, parts of the additional load after cracking could be transferred to the supports in the weak direction, leading to a two-way slab with a more even load distribution. Furthermore, this study showed that the difference in the effective support length decreased by the addition of steel fibres.

Considering the slabs with conventional reinforcement alone, the full length of the support was utilised in the strong direction. In the weak direction, the support length was only partially used. In slabs with conventional and steel fibre reinforcement, neither direction was fully utilized; however, the difference between the lengths utilised was reduced.

As a complement to the slab testing, the steel fibre reinforced concrete was characterised, both through uni-axial tests and three-point bending tests, and a significant scatter was observed. The scatter was closely connected to the number of fibres bridging the cracked zone in the tested specimens, which depended on the specimen geometry as well as the randomness of the fibre content in the fresh concrete. By normalizing the results with regard to the number of fibres in the fractured cross-sections and specimen geometry, the scatter was significantly decreased. Performing such an operation on the results from the uni-axial tests, it was possible to use numerical analysis to reproduce the results of the three-point bending test with reasonably good agreement within the range used for determining design parameters. This could be valuable as the determination of material parameters is dependent on the test method used. Furthermore, a similar approach might be utilised in relating other test methods to one another.

It is noteworthy that the scatter observed during the material testing was not witnessed on a structural scale, i.e. in the slabs tested. In contrast to the variation in response observed in both material test methods, virtually no scatter was observed in the tests of the fibre reinforced octagonal slabs. This implies that steel fibre reinforced concrete could be considered as a homogeneous material on the structural scale, in contrast to the material level. The main reason for this discrepancy is believed to be that the guiding notch controlled the fracture section of the material tests whilst the fractures in the slabs were, to a higher extent, controlled by weak points in the material, leading to a reduced scatter. Furthermore, the larger volume of concrete and the structural redundancy of having two load-carrying directions are also likely to contribute to the lower scatter. It should be stressed that even though no scatter was observed in terms of global load carrying behaviour, inhomogeneities in the slab might have affected the crack pattern.

Through a combination of experiments and numerical as well as analytical work on both the structural and material level, this thesis contributes to an improved understanding of the structural use of SFRC. These contributions highlight the benefits of steel fibre reinforced concrete, especially if applied to structures of complex geometry. Furthermore, the methodology discussed for the design of conventional reinforcement contributes to solving challenges associated with such concrete structures; thus facilitating a step forward for tailor-made concrete structures.

7.2 Suggestions for future research

To fully benefit from the ongoing development of production techniques, the need for a rational design method for non-standard concrete structures reinforced with conventional steel reinforcement has to be further developed. Ideally, a design method for steel fibre reinforcement might be implemented. Furthermore, the best reinforcement direction is not necessarily easy to find while considering concrete structures of complex geometry. Today, optimisation usually addresses the reinforcement amount or other structural parameters, e.g. deflection. In a design tool for automated production, it can be at least as important to optimise for the best possible production. Furthermore, shifting focus towards the entire design process, significant research efforts and software development are still needed to create platforms for cross-disciplinary cooperation.

As concluded in this thesis, steel fibre reinforced concrete can enhance the structural performance of concrete structures significantly. Despite the well-known potential, no widely accepted and comprehensive design recommendation has been proposed, in many cases hindering the application of the material. It is the author's belief that such a recommendation is crucial for further implementation of steel fibre reinforced concrete. Furthermore, in contemporary practice material characterising tests are needed to obtain the material behaviour of every steel fibre reinforced concrete mix. An international standard featuring definitions of standardised material classes and means of how to ensure that they are followed would be needed, analogous to the present practice of plain concrete.

Studying load redistributions in reinforced two-way slabs, with or without steel fibres, it was observed that the redistribution ability was favourably affected. However, the effect might be further quantified by using the experimental programme presented as a starting point. In future studies, parameters to be varied might include the ratio of conventional reinforcement in each direction (making the weak direction stronger), the effective depth of the slab and the steel fibre content.

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