Fibre Reinforced Profiled Mortar Joints for Precast Concrete Structures
Fibre Reinforced Profiled Mortar Joints for Precast Concrete Structures

By

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As previously said, a significant part of this thesis revolves around the material science of steel fibre reinforced mortar. Since my background is building engineering as opposed to material science, I faced many challenges. However, I was greatly guided by Steffen Grünewald in this area. I would therefore like to thank him for his excellent support, for his critical and informative feedback, and for his attention to the details of this thesis.

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Dijeannio A. Hobson
Rotterdam, August 2014
Summary

The precast concrete industry has been on the rise since its introduction in the early 20th century. As with many other industries, the precast concrete sector is growing and to continue this growth it is necessary that new innovations, ideas, technologies etc. are developed to improve the functionality of precast concrete elements. Many applications can be found nowadays for all different kinds of precast concrete elements. One such application for example is precast concrete shear walls. These walls may consist of elements of different shapes and sizes. The elements are connected at a joint. The joints are important for the structural functionality and have a large influence on the costs of the structure. Moreover, the joints are often the biggest drawback. To tackle this problem, many different technologies and methods have been and are being developed to improve the functionality of the joint, while taking the costs into account. One such method is the application of steel fibre reinforced mortar in a joint. The intention is to increase the shear strength of the joint by manipulating the material properties of the mortar by adding steel fibres. The steel fibres can also be accompanied by other fibres for the purpose of enhancing properties other than strength. Since not much is known of the effect of steel fibre reinforced mortar on the shear strength of the profiled mortar joints, research is needed.

This thesis focuses on the application of steel fibre reinforced mortar in profiled mortar joints of precast shear wall structures. A study was conducted based on experiments and analyses in order to investigate the effect of the fibres on the mechanical behaviour of the fibre reinforced mortar as well as the shear behaviour of the steel fibre reinforced profiled mortar joint.

To start the process, a literature review of existing fibres and research on profiled mortar joints were conducted to acquire knowledge that could be used for this thesis. Based on the review, two different types of fibres were selected for use in the mortar, i.e. straight smooth steel fibres and PVA fibres. When combined together they form a hybrid fibre reinforced composite. These two fibres were selected for the purpose of improving the strength properties and reducing shrinkage, respectively. Furthermore, the equal and shifted profiled mortar joint were considered for review. For the analysis, however, only the shifted profiled mortar joint was considered, since this joint has a higher load bearing capacity compared to the equal profiled joint.

To investigate the effect of the steel fibres on the material properties of the mortar, experiments were conducted. This was done in three phases. In the first phase, 12 different mixtures containing different (combinations of) steel and/or PVA fibre dosages were tested. The following properties were investigated: flexural strength, compression strength, shrinkage and workability. The intention was to gain a quick overview of the most promising mixtures. It was observed that the steel fibres affected the strength properties positively. This was less the case for the PVA fibres. Furthermore, shrinkage remained unaffected despite the presence of the PVA fibres. The tests that were conducted in the first phase did not provide any insight into the post-cracking behaviour of the material. However, literature states and it is known that fibres mostly affect the post-cracking strength. Therefore, a second phase of testing was carried out in order to investigate the flexural post-cracking strength, compression strength and splitting tensile strength of the fibre reinforced mortar. Four mixtures were selected for experimentation, following the first phase. Again it was observed that the steel fibres had a positive effect on the strength properties, whereas the PVA
fibres provided little to no contribution. In the third phase of testing, the execution method was altered. A mortar pump was introduced in order to investigate the pumpability of the fibre reinforced mortar. The same tests for flexural strength and compression strength were conducted as in phase 1. The pump tests were assumed to be successful, since the mortar was being pumped with little difficulty. Indeed, the compression strength of the mixtures was positively influenced, though marginal. However, the results of the experiments showed that the pump affected the flexural strength of the fibre reinforced mortar negatively. This was attributed to subpar fibre distribution and orientation as a result of pumping. Further research is required to solve the issue of the pump.

In practice and in theory, the uniaxial tensile strength would be used for describing the tensile behaviour of the cementitious composites. However, determining the uniaxial tensile strength from experiments is not an easy task as there many factors that can influence the outcome of the results. For this reason flexural tests are often conducted instead of uniaxial tensile tests, since these tests are more simple to carry and can be easily reproduced. However, to acquire the uniaxial tensile strength from the results of flexural tests, an inverse analysis procedure was required. This method uses an iterative approach to determine a parameter within a permissible margin of error of 10%. The multi-layer model, developed by Hordijk, is based on the inverse analysis approach. This model was used to carry out analyses. From this model the uniaxial tensile strength was obtained as well as parameters for the bilinear equivalent post-cracking strength, tensile elastic strain, critical crack-width and characteristic crack-width. These parameters define the stress/crack-width softening diagram that describes the post-cracking behaviour of the fibre reinforced mortar.

The parameters that were obtained from the multi-layer model were used as input for a finite element model of the beam specimens that were tested in the second phase of the experiments. The finite element model of the beam specimens provided good insight into the behaviour of the material. The intention was to further optimize the parameters obtained from the multi-layer model, with the goal of applying these parameters in the finite element model of the profiled mortar joint. The beam as well as the profiled mortar joint were modelled in ATENA 2D.

Lastly, a finite element analysis was conducted on the profiled mortar joint. Two mixtures containing two different steel fibre dosages (30 and 60 kg/m$^3$) were considered. The results showed that the fibre reinforced mortar increased the shear strength of the joint when compared with the reference mortar. However, no significant difference in the peak strength and pre-peak behaviour could be observed between the mixture containing 30 kg/m$^3$ and 60 kg/m$^3$. Only the post-peak strength was slightly higher for the mixture containing 60 kg/m$^3$. It was observed that the uniaxial tensile strength of the mortar played a larger role in the shear strength of the profiled mortar joint as opposed to the post-cracking strength of the mortar. Since both mixtures had approximately the same uniaxial tensile strength, the shear strength of the joint was similar for both mixtures. Furthermore, the crack formation of the finite element models of the profiled mortar joint showed some similarities with the tested specimens.
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1 Introduction

The precast concrete sector is growing and to accommodate this growth it is necessary that new innovations, ideas, technologies etc. are developed and implemented. An element that plays an important role for the structural design of precast concrete structures is the joint. The joints do not only influence the structural function, but also the cost aspects of precast concrete structures. Therefore, if precast concrete structures are to be used more in the future, special attention has to be paid to the joints. Though precast concrete has its advantages over cast-situ, the mortar joints are often the weak links in the structure. Furthermore, they are usually costly parts and a major drawback in the use of precast concrete structures. If precast concrete construction is to become a much more prominent method of construction in the future, then it is imperative that the joints need to be improved.

To start, questions can be asked concerning the joints. One of these questions is: “How can the performance of joints be influenced (or improved), in an affordable manner, as such that the entire behaviour of the precast structure is improved?”

One way to strengthen the joints is by adding fibres (e.g. steel fibres) to the mortar. Based on previous research on steel fibre reinforced concrete (SFRC) in mortar joints, it is expected that the joint will have an improved shear and tension behaviour [1]. However, this article and others that are found concerning this research often date back more than 20 years. Since then, concrete quality and the quality of the fibres have been improved significantly.

1.1 Research Outline

1.1.1 Background
The concept for this master's thesis project was developed with input from the ongoing research of Van Keulen [2] on high quality mortar joints. In his research the focus is directed on improving the shear strength and behaviour of horizontal smooth or vertical profiled joints by manipulating certain physical and material properties of the joint and structural elements. In addition, much attention is paid to the stiffness of the joint and the effect of stiffness. In this master thesis project a concept was developed for improving the shear behaviour of the joint through manipulation of the material properties of the mortar. Moreover, the material properties were altered by adding fibres to the mix.

1.1.2 Scope of the Research
This research focuses on determining the material properties of fibre reinforced mortar by execution of experiments and modelling. Subsequently, the results of the experiments and models are used as input for the analysis of precast concrete profiled mortar joints.

1.1.3 Research Question
The context of the research is based on the following main research question:
“How does fibre reinforced profiled mortar joints perform in terms of strength and stiffness in precast concrete shear wall structures?”

1.1.4 Research Objective
To answer the research question, the following main objective was set:
“Carry out research on the mechanical properties of fibre reinforced mortar and to investigate its effect on the strength and stiffness of a fibre reinforced profiled mortar joint.”

1.1.5 Research Strategy
Based on the research question and objective, an approach for the research was determined. It can be summarized as follows:

1. Literature review
2. Experimental Studies
3. Modelling the behaviour of fibre reinforced mortar using an inverse analysis approach and finite element analysis with ATENA 2D
4. Finite element modelling of the behaviour of fibre reinforced profiled mortar joints with ATENA 2D

1.2 Outline

Part A Literature Review
In this part of the thesis, literature on profiled mortar joints (shear key joints), steel and synthetic fibres is addressed. The properties and behaviour of profiled mortar joints and fibres are discussed.

Part B Experimental Studies
In this part of the thesis, the results of the experimental investigation of the material properties of the fibre reinforced mortar (FRM) are addressed. Moreover, strength properties, workability and shrinkage were investigated.

Part C Modelling Behaviour of Fibre Reinforced Mortar
In this part of the thesis, the behaviour of the different SFRM mixtures was modelled and analysed in order to simulate and investigate the tensile post-cracking behaviour. This was done in order to determine the tensile properties that were not obtained through experimentation. Furthermore, two approaches were taken into account for modelling the behaviour, i.e. inverse analysis and finite element analysis.

Part D FEM of profiled mortar joint
In this part of the thesis, the results of the finite element analysis (FEA) of a precast profiled mortar joint are addressed. The parameters obtained from the analysis of the FRM were used as input.

Part E Conclusions & Recommendations
Lastly, the report ends with conclusions and recommendations of the findings.
PART A
LITERATURE REVIEW
2 Loads and Failure Modes of Joints

In order to determine what is needed for the joints regarding the fibres, it is necessary to explore the behaviour of the joints and subsequent failure modes as a result of the shear forces.

2.1 Loads and Forces

Figure 1 displays an example precast structure with typical joints loaded by wind. Only the second and third joints (vertical mortar joints) are relevant for this master thesis. In these joints the fibres will play a larger role in the shear and tension capacity due to the compression diagonals that are created as a result of the keys (shifted and equal). Furthermore, the structure has mainly two functions; transfer of horizontal wind forces to the foundation and providing stability. The horizontal forces are transferred through shear and bending. The magnitude of the forces transferred in a joint and the deflection of the entire structure depend on the stiffness, concrete strength and properties. The figure illustrates how the joints transfer the shear forces corresponding to the orientation of the load. When the strength of the concrete is exceeded, failure occurs. Failure modes correspond with the flow of forces in the joint, and strongly depends on the concrete properties and configuration of the joint. The following subchapter expands on this.

Figure 1: A: Example of Precast Structure consisting of large concrete panels, B: Plain and Profiled joints

2.2 Failure Modes

The behaviour of large wall panels, assuming that the profiled joints are part of this structure, depends largely on the mechanical properties of the joints. It is evident that fibres can improve the mechanical properties of these joints. However, before it can be decided what improvements are needed and the best suited fibres corresponding to these improvements, it is necessary to comprehend how the joints behave under the present load and which failure modes and corresponding mechanical properties are relevant. As mentioned previously, the vertical joints play an important role in transferring lateral loads on the structure through shear forces. Therefore, the relevant mechanical property of the joints in this project concerns the shear strength. The following questions concerning the mortar joints can be asked:

- Is high strength and/or ductility most important?
- Or, is crack-resistance most important?
To answer these questions one has to look at what is governing. To illustrate this, the equal profiled joint is investigated. Acting on the joints are shear (T) and normal forces (N), see Figure 2. Failure due to these forces can result in the modes shown in Figure 3.

![Figure 2: Shear and normal forces on the equal profiled joint](image)

Figure 2 displays four different failure modes that correspond with four different properties of the concrete; shear, compression, tension and shrinkage (possibly) respectively[3]. Fibres can improve these different properties of the concrete at different extents. It may be determined which failure is governing, and therefore the corresponding property can be improved.

![Figure 3: Failure Modes of Vertical Shear Key Joints](image)

Figure 3: Failure Modes of Vertical Shear Key Joints; a: shearing, b: crushing, c: compressions diagonal tension cracking, d: slipping [3]

- a): the joint failed completely in shear. In the case of FRC, the shear strength is relevant in this case.
- B): The joint failed due to crushing. In the case of FRC, compressive strength is relevant in this case
- c): the joint failed due to diagonal tensile cracking. In the case of FRC, the tensile or splitting tensile strength is relevant in this case.
- d): the joint failed due to slip. Friction between concrete mortar and concrete surface is insufficient. This could be attributed to drying shrinkage or poor execution.

If pre-stressing is present then it is highly likely that, in the case of excess loading, either modes a, c or a combination would occur. This can be observed in Figure 4. This figure displays the results of tests on profiled mortar joints. It shows that these modes are likely to occur provided that the joints are executed satisfactorily. It can be noticed that a combination of these two modes can also occur. Specimen 5 (shifted profile without fibres) and 9 (equal profile without fibres) show a combination of diagonal tension cracks and shear cracks in one key, while specimen 10 (equal profile with fibres) shows both failure modes. However, the modes are not combined in one key but have occurred in the keys independently. Specimen 12 shows diagonal cracks (bottom key) and shear cracks (top and middle key). It can also be noticed that specimen 10 and 12 show dominant cracks. This is particularly
evident in specimen 10. This can possibly be attributed to the effect of the fibres on the concrete mortar properties. The fibre content, distribution and orientation as well as the properties of the fibres will affect the formation and types of cracks and strength of the matrix and ductility of the matrix. These shear cracks can likely be attributed to lower fibre count at the location of the crack or the orientation of the fibres, which could indicate that fibre distribution and thus execution is important.

Figure 4: Failure mode of the joints; A: specimen 5, B: Specimen 9, C: specimen 10, D: Specimen 12

2.3 Evaluation

- In general there are four failure modes for the shear keys; shearing, crushing, diagonal cracking and slip.
- Shearing and diagonal tensile cracking appears to be governing failure modes for the shear key joints. These failure modes correspond to strength properties of the mortar, i.e. tension and shear strength.
- The first crack strength corresponding to these properties of the mortar can be increased by adding the fibres.
- However, these fibres should possess the right properties in order to increase the strength of the mortar.
- Furthermore, adding fibres may increase the ductility of the joint.
3 Profiled Mortar Joints

This chapter addresses the shear behaviour of the joints.

3.1 Behaviour of Shear Walls with Profiled Mortar Joints

Rizkalla et al. [5] investigated multiple shear key connections. In this study, several horizontal shear key joints used in precast concrete shear wall panels were tested. The behaviour and capacity of the shear key connection was determined.

3.1.1 Details

Figure 5 shows the overall dimensions of the specimen.
- Configuration of the keys
- Magnitude of pre-tensioning force normal to horizontal connection
- Two different pre-tensioning forces 2 MPa and 4 MPa

Specifics:
- Dry pack grout
  - Sand-Cement -Water ratio = 2:10:0.2
  - Portland Cement Type 10 was used
- Dimensions of the specimens:
  - Depth 200 mm
  - Width 1880 mm
  - Height 2300 mm
- Two levels of pre-loading applied
  - 2 MPa
  - 4 MPa

![Figure 7: Test specimen details](image)

### 3.1.2 Results

Figure 8 show the typical load-slip curve of the shear key connection.

![Figure 8: Typical load-slip behaviour of multiple shear key connections](image)

Various limit states can be recognized in the graph for the connections:
- Behaviour prior to cracking
- The maximum load
- The ultimate shear resistance at large slip

The cracking load, $V_{cr}$, was determined as the load corresponding to the initiation of diagonal tensile cracks in the drypack shear keys. The maximum load, $V_m$, is the peak load as can be seen in the graph. The ultimate shear resistance, $V_u$, is the load corresponding to a slip of 5 mm at the connection.

Failure modes as discussed in chapter 2 can be clearly observed in Figure 9. The cracking behaviour of both the large and small shear key connections is virtually the same. Further compression of the specimen resulted in crushing of the struts, which formed between the diagonal cracks and by slip along the crack surface.

![Figure 9: Typical crack patterns at failure](image)

The behaviour of the connections is shown in Figure 10. From the curve it can be observed that the difference between the two shear key connections had little to no effect on the behaviour or capacity of the connection. In fact, it appears that the increase is attributed only to the pre-tensioning force. Furthermore, it appears that debonded\textsuperscript{1} shear key connections (e.g. 3SK4B) experience a slightly larger increase in peak load. For both debonded shear key connections the curve is similar.

\textsuperscript{1} Debonded shear key joints were prepared in such a way that the adhesion between concrete elements and dry-pack grout at the interface was absent.
Figure 10: Effect of key configuration and level of pre-load

A comparison of the shear connections under a pre-tensioning force of 4 MPa is shown in Figure 11. The difference between the shear keys and the plain surface joint can be clearly observed. The shear key greatly enhances the shear capacity compared to the plain surface connection. An increase of approximately 60 percent was recorded. Furthermore, the large shear key connection appears to possess a slightly better ductile behaviour. Regarding the ultimate shear strength, an increase as much as 25 percent was recorded compared to the plain surface joint.

Figure 11: Effect of shear keys

This increase in shear capacity is attributed to the increase in confinement, and consequently the tensile resistance of the dry/pack, provided by the higher pre-tensioning load. The ultimate shear resistance of the large and small shear key connections were increased by 80 and 50 percent, respectively, when the magnitude of the pre-tensioning load was increased from 2 to 4. The results show that the shear capacity does not increase by the same ratio as the increase of the pre-tensioning load. This suggests that the behaviour of the shear key connection cannot be fully
described by the simple shear friction theory, which states that the shear resistance is directly proportional to the level of stress normal to the connection.

<table>
<thead>
<tr>
<th>Joint configuration</th>
<th>Specimen mark</th>
<th>Cracking load, $V_{cr}$, (kN)</th>
<th>Maximum load, $V_{m}$, (kN)</th>
<th>Ultimate shear resistance, $V_{ur}$, (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Large key</td>
<td>1L.K2</td>
<td>500</td>
<td>569</td>
<td>418</td>
</tr>
<tr>
<td></td>
<td>K3K4</td>
<td>800</td>
<td>867</td>
<td>688</td>
</tr>
<tr>
<td></td>
<td>3L.K4B</td>
<td>1000</td>
<td>1088</td>
<td>694</td>
</tr>
<tr>
<td>Small key</td>
<td>2SK2</td>
<td>500</td>
<td>559</td>
<td>419</td>
</tr>
<tr>
<td></td>
<td>1SK4</td>
<td>803</td>
<td>884</td>
<td>622</td>
</tr>
<tr>
<td></td>
<td>3SK4B</td>
<td>807</td>
<td>893</td>
<td>648</td>
</tr>
<tr>
<td>Plain surface</td>
<td>1N.K4</td>
<td>503</td>
<td>540</td>
<td>507</td>
</tr>
</tbody>
</table>

Note: 1 kN = 0.225 kip.

Figure 12: Summary of measure shear strengths

### 3.2 Behaviour Shear Walls with Steel Fibre Reinforced Profiled Mortar Joints

Abdul-Wahab [1] investigated the effect of steel fibres on the shear behaviour of shear walls with SFRC joints. Experimental results were compared with predicted values based on modified forms of the shear-friction methods proposed in the ACI Building Code and BS 8110. A theoretical method was proposed that takes into account the combined effect of the dowel action of reinforcement and the improved concrete shear resistance due to the fibres.

It is stated that substantial increase in shear and tensile (diagonal tension) strength of concrete can be obtained by including steel fibres in the matrix. Aside from the benefits of the inclusion of steel fibres mentioned previously, the steel fibres increase the shear-friction strength, which is the governing material property in vertical joints.

#### 3.2.1 Details

- Three type of joints were considered:
  - Plane
  - Castellated (shear keys)
  - Grooved

- Materials used:
  - Grout
    - Ordinary Portland Cement
    - Natural Sand
    - Coarse aggregates max 19 mm
  - Cement-sand-aggregate ratio; 1:1.5:3 (by weight) for panels and grout
  - Water Cement Ratio (w/c): 0.5
  - Reinforcement
    - 10 mm
    - Yield Strength First Series: 450 MPa
    - Yield Strength Second Series: 509 MPa

- Dimensions:
  - Height: 700 mm
  - Width: 610 mm
  - Depth: 150 mm
Transverse reinforcement is present in the joint. This increases the strength and ductility.

<table>
<thead>
<tr>
<th>Steel Fibre</th>
<th>Type I</th>
<th>Type II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hooked-end smooth drawn wire</td>
<td>30</td>
<td>25</td>
</tr>
<tr>
<td>Plain straight</td>
<td>0.5</td>
<td>0.4</td>
</tr>
<tr>
<td>Aspect Ratio</td>
<td>60</td>
<td>62.5</td>
</tr>
<tr>
<td>Yield Strength [MPa]</td>
<td>1177</td>
<td>1177</td>
</tr>
</tbody>
</table>

From Figure 14 it can be observed that the addition of steel fibres does not increase the compressive strength of the composite.
3.2.2 Results

Figure 16 shows the load-displacement curves of the three different joints. It can be clearly observed that increasing the fibre content may significantly increase the ultimate shear strength of the joints. Also, the shear strength increased linearly for the shear key joints, see Figure 17. In addition, the shear key joints appear to benefit the most from the addition of the steel fibres.
Figure 18 shows the results of the tests conducted. With 1 percent steel fibre, an increase in shear strength of 54 percent was observed for the shear key joint.

![Figure 17: Effect of fibre content](image1)

![Figure 18: Summary of experimental and predicted results](image2)
4 Fibres

It is evident that fibres are being applied more often in concrete nowadays e.g., High Performance Concrete (HPC), Ultra High Performance Concrete (UHPC), Self-Compacting Concrete etc. Moreover, as a result of the fibres these concrete types have experienced an improvement not only in strength, but durability, increased resistance to various external agents, high rate of hardening, better aspects, etc [6].

With that said, to better comprehend fibre reinforced composites (FRC) and to understand how the fibres can be used to optimally improve the performance of the mortar joints in precast structures, attention has to be given to the fibres (i.e. types, material- and mechanical properties, and behaviour) separately. The aim is to find suitable type(s) of fibre(s) for the joints that will be experimented on in this research.

4.1 Starting Point

Often rigorous mechanical mixing is required to blend the fibres with fine aggregates, cement and any filler materials present. In addition, only robust types of fibres are capable of withstanding the bending, impact and abrasive effects inherent in rigorous mechanical mixing. For this reason often only steel, carbon and some synthetic fibres such as polypropylene, polyethylene, polyester, and nylon are considered for reinforced concrete in which mechanical mixing is applied [7]. Steel fibres are most commonly used, but fibrillated polypropylene, and single forms of polypropylene, polyethylene, polyester can be incorporated into the concrete mix. Furthermore, other fibre types e.g. glass and natural fibres are often not durable and need extra (often expensive) precautions to make these fibres suitable for the concrete environment and for mixing.

There are many different types of fibres in commerce with different properties. Banthia (2008) has discovered that steel fibres remain the most used fibres (50 % of total tonnage used), followed by polypropylene (20 %), glass (5 %) and other fibres (25 %). Since there are so many fibres available for use in fibre reinforced concrete, it was decided to narrow down the options based on availability and common applications. This lead to the following three fibre types, i.e. steel (straight thin wire fibre), polypropylene, nylon, and polyvinyl alcohol (PVA). Straight thin steel wire fibre is considered due to its flexibility and bending capabilities which is desirable for the execution method as mentioned previously. Moreover, it suits the boundary conditions in terms of physical characteristics, see Chapter 6. In addition, nylon and PVA have also been considered due to its similar effect on concrete properties compared to polypropylene fibres. These fibres are expanded on in the following chapters.

4.2 Steel Fibres

Steel fibre-reinforced concrete uses thin steel wires that is mixed with the cement, see Figure 19. This imparts the concrete with greater structural strength, reduces cracking and helps protect against extreme cold.

The typical content for fibres by volume ranges from 0.25% to 2%. Higher contents will generally result in poor workability and fibre distribution [8]. Flexural toughness and ductility of concrete can be enhanced by high elastic Modulus steel fibres. These contribute mainly after the matrix has
cracked. However, the high stiffness of steel fibres can cause micro-defects such as voids and honeycombs during placing as a result of improper consolidation at low workability levels [9].

Figure 20 shows recommended values for the maximum fibre contents without encountering workability and fibre balling issues. Assuming that the fibres in the second series will have an aspect ratio in the range 60-80 and that the FR mortar will be pumped, then the maximum fibre content for steel fibres according to this table can be at least $120 \text{ kg/m}^3$. This is just an indication however. Note that the maximum coarse aggregate for the Cuglaton Tiksomortel K70 is approximately 0.5 mm. Furthermore, these values concern steel fibres with hooked ends. In addition, the type of pump is also relevant for these values. For other fibre types, the pattern should be similar, but with modifications to the actual numbers to reflect differences in fibre profile, surface texture and nature of end anchorage [7].

![Image of steel fibres](image)

Figure 19: Example of steel fibres [10]

<table>
<thead>
<tr>
<th>Max. coarse aggregate size — mm</th>
<th>Steel fibre aspect ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>60</td>
</tr>
<tr>
<td></td>
<td>75</td>
</tr>
<tr>
<td></td>
<td>100</td>
</tr>
<tr>
<td>Normal</td>
<td>Pumped</td>
</tr>
<tr>
<td>Normal</td>
<td>Pumped</td>
</tr>
<tr>
<td>Normal</td>
<td>Pumped</td>
</tr>
<tr>
<td>4</td>
<td>160</td>
</tr>
<tr>
<td>8</td>
<td>125</td>
</tr>
<tr>
<td>16</td>
<td>85</td>
</tr>
<tr>
<td>32</td>
<td>50</td>
</tr>
</tbody>
</table>

Figure 20: Maximum recommended steel fibre contents for Steel Fibre Reinforced Concrete (SFRC) [kg/m$^3$] [7]

Advantages:
- High Tensile strength: 0.5 – 2 GPa
- High Modulus of elasticity: 200 GPa
- Ductile/plastic stress-strain characteristic
- Low creep
- Durable, particularly in high-temperatures
- Substantial improvement in resistance to impact and greater ductility of failure in compression, flexure and torsion

Disadvantages:
- Lack of corrosion resistance. This is particularly the case in normal steel fibres (carbon steel) in exposed concrete situations where spalling and surface staining occurs.
This can be solved by using fibres coated with corrosion-resistant alloys. This, however, may have consequences regarding costs.

4.3 Synthetic Fibres

Synthetic fibres commonly consist of polypropylene, polyester, nylon, polyvinyl alcohol, polyacrylonitrile (acrylic) or polyolefin in the form of separate monofilaments, bundles monofilaments or, in the case of polypropylene, more commonly in the form of multifilament fibrillated strands. Concrete mixes containing these fibres need to be proportioned with a larger than normal mortar fraction. This is done by reducing the coarse aggregate volume fraction and increasing the fine aggregate or adding fly ash, silica fume or slag [7]. Typically, two different ranges of fibre volume are applied; low-volume percentage (0.1 to 0.3%) and high-volume percentage (0.4 to 0.8%) [8].

The following table shows properties of different synthetic fibres commercially available for fibre reinforced concrete. Note that only nylon and polypropylene are regarded for this thesis.

Table 2: Properties of synthetic fibre types [11]

<table>
<thead>
<tr>
<th>Fiber type</th>
<th>Equivalent diameter, in. x 10^3</th>
<th>Specific gravity</th>
<th>Tensile strength, ksi</th>
<th>Elastic modulus, ksi</th>
<th>Ultimate elongation, percent</th>
<th>Ignition temperature, degrees F</th>
<th>Melt. oxidation, or decomposition temperature, degrees F</th>
<th>Water absorption per ASTM D 570, percent by weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acrylic</td>
<td>0.5-4.1</td>
<td>1.10-1.18</td>
<td>59-143</td>
<td>2000-2800</td>
<td>7.5-20.0</td>
<td>—</td>
<td>450-475</td>
<td>1.0-2.3</td>
</tr>
<tr>
<td>Armafil I</td>
<td>0.47</td>
<td>1.44</td>
<td>425</td>
<td>9000</td>
<td>4.4</td>
<td>high</td>
<td>900</td>
<td>4.3</td>
</tr>
<tr>
<td>Armafil II</td>
<td>0.40</td>
<td>1.44</td>
<td>340</td>
<td>17,000</td>
<td>2.3</td>
<td>high</td>
<td>900</td>
<td>1.2</td>
</tr>
<tr>
<td>Carbon, PAN H3F</td>
<td>0.30</td>
<td>1.6-1.7</td>
<td>160-440</td>
<td>55,100</td>
<td>0.5-0.7</td>
<td>high</td>
<td>752</td>
<td>nil</td>
</tr>
<tr>
<td>Carbon, PAN HT1</td>
<td>0.35</td>
<td>1.6-1.7</td>
<td>500-580</td>
<td>33,400</td>
<td>1.0-1.5</td>
<td>high</td>
<td>752</td>
<td>nil</td>
</tr>
<tr>
<td>Carbon, pitch OP</td>
<td>0.39-0.51</td>
<td>1.6-1.7</td>
<td>70-115</td>
<td>4000-5600</td>
<td>2.0-2.4</td>
<td>high</td>
<td>752</td>
<td>3.7</td>
</tr>
<tr>
<td>Carbon, pitch HP</td>
<td>0.35-0.70</td>
<td>1.80-2.15</td>
<td>220-450</td>
<td>22,000-76,000</td>
<td>0.5-1.1</td>
<td>high</td>
<td>932</td>
<td>nil</td>
</tr>
<tr>
<td>Nylon</td>
<td>0.80</td>
<td>1.14</td>
<td>140</td>
<td>750</td>
<td>20</td>
<td>—</td>
<td>392-450</td>
<td>2.8-5.0</td>
</tr>
<tr>
<td>Polyester</td>
<td>0.78</td>
<td>1.34-1.39</td>
<td>83-160</td>
<td>2500</td>
<td>12-150</td>
<td>1100</td>
<td>495</td>
<td>0.4</td>
</tr>
<tr>
<td>Polyethylene</td>
<td>1.0-4.00</td>
<td>0.92-0.96</td>
<td>11-85</td>
<td>725</td>
<td>3-80</td>
<td>—</td>
<td>273</td>
<td>nil</td>
</tr>
<tr>
<td>Polypropylene</td>
<td>—</td>
<td>0.00-0.91</td>
<td>20-100</td>
<td>500-760</td>
<td>15</td>
<td>1100</td>
<td>330</td>
<td>nil</td>
</tr>
</tbody>
</table>

*Not all fibre types are currently used for commercial production of FRC.
Polypropylene based, high modulus.
Polyvinyl alcohol based, high modulus.
Polyvinyl alcohol based, high tensile strength.
**Polyethylene based, general purpose.
Polypropylene based, high performance.
Crimped pitch based, high performance.
Data listed is only for fibre commercially available for FRC.
Metric equivalents: 1 in = 25.4 mm; 1 ksi = 6895 MPa; (degrees F - 32)/1.8 = degrees C.

4.3.1 Polypropylene Fibres

The use of polypropylene fibres at low volume fractions provides concrete with improved performance characteristics at reasonable cost. It is considered to be an effective method for improving the shrinkage cracking characteristics, toughness and impact resistance of concrete materials. Polypropylene fibres are polymer based fibres and can consist of fibrillated strands or monofilament strands, see Figure 21 and Figure 22. The fibrillated are intended to separate during mechanical mixing into smaller strands comprising a few longitudinal filaments cross linked by transverse filaments to form an open lattice that is penetrated by the matrix mortar, thus creating a mechanical bond [7].

Monofilament polypropylene may be crimped to improve bond with cement or coated to improve workability and dispersability. These are available in bundles that disperse in concrete during mixing.
The mixture-stiffening and workability reducing effect of polypropylene fibres is caused by their large surface area and depends on the fibre amount and fibre/strand length. When compared to the steel fibres, the synthetic fibres have an advantage regarding the workability. This is backed by the claim that while they are not as strong as steel, they do help improve the cement pumpability by reducing clumping [7]. This is due to its fineness and relatively low stiffness. Other advantages and disadvantages are mentioned below.

Figure 21: PP fibres in monofilament form and fibrillated form [source: made-in-china.com PP monofilament Fibre, PP Mesh Fibre (JT-6)]

State of the Art Report by the ACI committee 544 [11] states that in general the addition of polypropylene fibres at different quantities has no effect on the compressive strength and that the minor differences noticed are a result of the different experimental work. In addition, they can be cause by variation in the actual air contents of the hardened concrete and the different in their unit weights. However, it has a significant effect on the failure mechanism, particularly for high strength concrete. The matrix shows more of a ductile behaviour.

The following table shows manufacturers recommended strand lengths for fibrillated polypropylene.
Table 3: Manufacturers recommended strand lengths for fibrillated polypropylene [7]

<table>
<thead>
<tr>
<th>Aggregate&lt;sup&gt;a&lt;/sup&gt; size — mm</th>
<th>Strand&lt;sup&gt;b&lt;/sup&gt; length — mm</th>
<th>Aggregate&lt;sup&gt;b&lt;/sup&gt; size — mm</th>
<th>Strand&lt;sup&gt;b&lt;/sup&gt; length — mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>6–13</td>
<td>6</td>
<td>19</td>
</tr>
<tr>
<td>10–16</td>
<td>19</td>
<td>13</td>
<td>38</td>
</tr>
<tr>
<td>16–25</td>
<td>38</td>
<td>19</td>
<td>57</td>
</tr>
<tr>
<td>19–38</td>
<td>51</td>
<td>25+</td>
<td>63</td>
</tr>
</tbody>
</table>

<sup>a</sup> Fibermesh flat strand.<br>
<sup>b</sup> Forta twisted strand.

Advantages:
- Improves freeze-thaw resistance. Insensitive to temperature differences due to low expansion rate. This helps prevent cracking.
- Improves impact resistance
- Advantageous in high strength concrete application in which fire and high temperatures play a prominent role, which is particularly beneficial in high density concrete where high stresses behind the surface can occur due to the fire. Moreover, these high stresses can lead to spalling of the concrete.
- Increases resistance to plastic shrinkage during curing
- Chemically inert
- Corrosion resistant to alkaline environment of concrete

Disadvantage:
- The mechanical bond as mentioned above is desirable, because monofilament fibres have inherent weak bond with the mortar matrix. This is because polypropylene is hydrophobic and therefore not easily wetted by cement paste to naturally develop an adhesive bond. However, proprietary surface treatments are used by some manufacturers to try and improve adhesive bond.
- Low melting points and high combustibility
- Though fibres are tough, they have low tensile strength and low stiffness (young’s Modulus). In addition, they have a plastic stress-strain characteristic
- Creep prone

4.3.2 Nylon/PVA

Nylon

Nylon is a polymer based fibre and can be separated into two types; nylon 6 and nylon 6,6. Nylon 6,6 is more commonly used as fibres for concrete applications, see Figure 23. According to literature, nylon is particularly effective in imparting impact resistance and flexural toughness to the matrix. In addition, it was also shown that nylon fibres lead to significant improvement in toughness, ductility, and control of cracking at contents ranging from 0.5 to 3 percent by volume [11]. Nylon has been shown to be particularly effective in sustaining and increasing the load carrying capability of following the first crack. However, the results concerning compressive and splitting tensile strength have been conflicting. Some research has shown the effect of nylon fibres on the compressive and splitting tensile strength to be negligible while another has shown decreases in the compressive strength of the mortar with increasing fibre content. For this test nylon fibre with a length of 13 mm
and 15 denier\(^2\) material was used at a fibre content up to 1 percent by volume. Furthermore, another study shows that the addition of nylon at 2.4 percent by volume does not significantly increase the splitting tensile strength. The comparative study between nylon- and polypropylene fibres expanded on below shows improvements as result of both nylon- and polypropylene fibres. The nylon and polypropylene fibres considered for this research possessed a length of 19 mm [13]. According to literature the effectiveness of low Modulus, synthetic fibres such as nylon to reinforce concrete and enhance its properties is primarily governed by the fibre/cement interface, fibre geometry (high aspect ratio), and fibre distribution. In other words, the difference can be related to the difference in fibre length used in the studies. Lastly, one study has shown the ability of nylon fibres to reduce concrete shrinkage by as much as 25 percent as measured by length. Fibre content ranging between 1 to 3 percent by volume of concrete was applied in this case.

![Figure 23: Nylon fibre medium denier with superior finish [source: www.nycon.com]](source: www.nycon.com)

Advantages:
- Heat Stable
- Resistant to a wide variety of materials
- Improves freeze-thaw resistance
- Increase resistance to plastic shrinkage during curing
- High elongation
- High abrasion resistance
- Relatively inert
- Effective in bestowing impact resistance and flexural toughness to matrix
- Effective in sustaining and increasing the load carrying capacity of concrete following first crack provided the fibres are long enough.

Disadvantages:
- Hydrophilic, which could increase mixing time thereby decreasing workability. Though disadvantageous for mixing, this particular property leads to better chemical bond with mortar matrix. Note that the bond of polypropylene fibres is only mechanical( higher bond for fibrillated fibres)
- Creep prone

\(^2\) Denier or den is a unit of measure for the linear mass density of fibres. It is defined as the mass in grams per 9000 meters. 1 denier =1 gram per 9000 m or 0.111 mg/m (Wikipedia)
PVA fibres have similar properties to nylon fibres. The most significant difference between the two is the Young’s Modulus. PVA fibres possess a much higher modulus of elasticity. For more on PVA fibres, see Appendix B – Kuralon PVA.

4.4 Synthetic Fibre versus Steel Fibre

4.4.1 Comparison

Many studies have been conducted on fibre reinforced composite (concrete, mortar) and the advantageous and disadvantages have been duly noted. Bekaert, a leading corporation in the use of fibres, have collected the pros and cons of using these fibres [12]. These are presented in the tables below.

<table>
<thead>
<tr>
<th>Property</th>
<th>Micro Synthetic Fibre Reinforced Concrete</th>
<th>Steel Fibre Reinforced Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>strength and toughness</td>
<td>***</td>
<td>***</td>
</tr>
<tr>
<td>energy absorption</td>
<td>***</td>
<td>***</td>
</tr>
<tr>
<td>resistance to plastic shrinkage</td>
<td>***</td>
<td>***</td>
</tr>
<tr>
<td>Freeze-thaw resistance</td>
<td>***</td>
<td>***</td>
</tr>
<tr>
<td>resistance to spalling in a fire</td>
<td>*</td>
<td>***</td>
</tr>
<tr>
<td>impact resistance</td>
<td>*</td>
<td>***</td>
</tr>
<tr>
<td>resistance to drying shrinkage</td>
<td>***</td>
<td>***</td>
</tr>
<tr>
<td>structural reinforced concrete</td>
<td>***</td>
<td>***</td>
</tr>
</tbody>
</table>

Figure 24: Comparison of Micro Synthetic and Steel Fibre Reinforced Concrete[12]

This table complements the advantages and disadvantages mentioned in previous chapters. Macro synthetic and steel fibres have a structural use. This is not the case for micro synthetic fibres. Micro synthetic fibres may complement steel or macro synthetic fibres in a hybrid fibre composite. A. Laning [14] further points out that synthetic fibres are not suitable as (primary) structural reinforcement in concrete, because it adds little or no strength to the composite.

Figure 25 presents the quantitative differences in the properties of steel and synthetic fibres. Note that all fibres in EU must comply with the following standards

- EN 14889-1: Steel fibres – Definitions, specifications and conformity
- EN 14889-2: Polymer fibres - Definitions, specifications and conformity
<table>
<thead>
<tr>
<th>Material</th>
<th>Concrete</th>
<th>Steel Mesh / Steel fibre</th>
<th>Micro / Macro Polymer Fibre</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thermal Expansion coefficient α</td>
<td>$12 \times 10^{-6}$°C</td>
<td>$12 \times 10^{-6}$°C</td>
<td>$1.5 \times 10^{-6}$°C</td>
</tr>
<tr>
<td>(Shrinkage for a temperature decrease of 30°C on a 50 mm long fibre)</td>
<td>0.018 mm</td>
<td>0.018 mm</td>
<td>0.23 mm</td>
</tr>
<tr>
<td>Creep behaviour in tension (Tg glass transition temperature)</td>
<td>+377°C</td>
<td>-20°C</td>
<td>-165°C - does not reinforce</td>
</tr>
<tr>
<td>Melting Point (°C)</td>
<td>1500°C</td>
<td>165°C</td>
<td>165°C</td>
</tr>
<tr>
<td>Young's Modulus</td>
<td>30,000 MPa</td>
<td>210,000 MPa</td>
<td>3,000 – 10,000 MPa</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>7,850 kg/m³</td>
<td>3,000 – 6,000 MPa</td>
<td>200 – 650 MPa</td>
</tr>
<tr>
<td>Density</td>
<td>2,400 kg/m³</td>
<td>4,000 kg/m³</td>
<td>910 kg/m³</td>
</tr>
<tr>
<td>Resistance to UV light</td>
<td>in concrete, and cracks &lt; 0.2 mm</td>
<td>in concrete, and cracks &lt; 0.2 mm</td>
<td>degradation will occur</td>
</tr>
<tr>
<td>Corrosion resistance</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Typical length of fibres</td>
<td>30–50 mm</td>
<td>0.5 – 1.0 mm</td>
<td>micro: 6–30 mm</td>
</tr>
<tr>
<td>Typical diameter of fibres</td>
<td></td>
<td></td>
<td>macro: 30–45 mm</td>
</tr>
<tr>
<td>Bekairet brands</td>
<td>Dramix®, Wiremix®</td>
<td>micro: Duomix®</td>
<td>macro: Symmix®</td>
</tr>
<tr>
<td>CE label is compulsory in EU in accordance with</td>
<td>EN 14188-1</td>
<td>EN 14188-2</td>
<td></td>
</tr>
</tbody>
</table>

**Figure 25: Steel Fibre vs. Synthetic Fibre [12]**

**Figure 26: Reinforcement properties of the composite material**
Plastic Shrinkage Resistance of Micro Polymer Fibre Reinforced Composite

According to studies conducted by Bekaert, the development of the young’s Modulus and the difference between Young’s Modulus of concrete and synthetic fibres follows the curve displayed Figure 27.

![Figure 27: Young’s Modulus concrete vs. polymer fibres](image)

It is stated that polymer fibres have only a plastic reinforcement effect in the first 24 hrs when their Young’s Modulus exceeds the fresh concrete Young’s Modulus. If micro synthetic fibres are used, then any crack that might appear will be bridged by thousands of micro fibres. In the composite material, a reinforcement effect can only be obtained when the reinforcing material displays a higher Young’s Modulus than the basic material to be reinforced such as concrete. Therefore, micro polymer fibres reinforced the very young and still plastic concrete with ease, attributed to the dense network of fibres.

Drying Shrinkage

After 24 hours, the mechanical properties of concrete multiply and Young’s Modulus exceeds that of the synthetic fibres. The Young’s Modulus of a fibre plays a major role in crack control. The higher the Young’s Modulus, the better the control of the cracks in terms of crack length and crack opening. Due to their low Young’s Modulus, macro synthetic fibres require large crack widths before they develop any useful stress in tension. For this reason, larger crack openings in aged and cracked concrete structures with macro synthetic fibres can be observed compared to steel fibres. Consequently, the deformation of the structure may be (too) significant. This can also be explained by looking at the constitutive relation between the deformation and the stresses, see Figure 28.
In Appendix B – Kuralon PVA, it is shown that PVA have a Young's Modulus of approximately 29 GPA. Therefore, it can be assumed that PVA fibres may affect the drying shrinkage of the composite.

4.4.2 Evaluation

The tables show that steel fibres have greater advantages that suits structural usages. However, micro synthetic fibres appear to be a good candidate for improvement of the fresh state properties of the concrete, i.e. reduction of plastic shrinkage.

The benefits of synthetic fibres consist of the following:

- They provide resistance to early plastic shrinkage unlike structural reinforcement or steel fibres
- Increase impact- and abrasion resistance and toughness or ductility.

However, synthetic fibres are often not the preferred choice of reinforcement for structural use due to the following reasons:

- Polymer fibres melt at 165 degrees Celsius. This may be advantageous in reducing spalling of concrete during fires. However, in the case of macro fibres intended for structural use, any reinforcing effect of the fibres fades away as the temperature rises.
- The Young's Modulus ranges between 3 – 10 GPa, except in the case of PVA fibres. This is very low and may be insufficient to reinforce concrete material with a Young's Modulus of 30 GPa.
- Macro polymer fibres are affected by creep. At ambient temperature, the polypropylene is typically visco-elastic, with significant creep. In addition, the deformation of the fibre is not only time-dependent, but also temperature-dependent, see figure below.

---

Figure 28: Constitutive Equation, deformation and Young's Modulus [12]

---

3 Structural Use: addition of fibres is intended to contribute to the load bearing capacity of the composite element.
If special measures are not taken, then creep of the fibre can lead to:

- Unsustainable crack widths (which will widen, over time, under constant loading,) thereby adversely affecting the durability of the concrete, serviceability etc.
- Creep rupture of the fibre even at stress levels corresponding to the serviceability limit state.

Research has shown that because of these drawbacks, synthetic fibres provide no significant improvement of the compressive, tensile and flexural strengths, modulus of elasticity, and splitting tensile strength.

### 4.5 Influence of the Fibres

The fibres are highly influential in the post-cracking behaviour of concrete due to its crack arresting effect on a macro – and micro level, see Figure 30.

Influence of the fibres consists of the following:

- The addition of fibres to concrete causes an increase of the fracture toughness due to the dissipation of energy through toughening mechanisms, such as fibre crack bridging, fibre bending, and fibre pull-out or de-bonding.
- Short fibres delay the formation of randomly distributed micro-cracks, consequently contributing in delaying the damage and strain localization phenomena. It in turns enhances resistance, first crack strength and ductility by means of the increase of energy dissipation.
When cracks propagate and micro-cracks localize in macro-cracks, long fibres can bridge the opposite crack faces and transfer the present tensile forces. This leads to an improvement in bearing capacity and ductility of the structure.

Strength and ductility of the matrix will increase by application of strong and high elastic fibres. Strong high elastic fibres will transfer a higher portion of the load prior to and after cracking has occurred thereby increasing the tensile strength and ductility of the matrix.

If improvement of the material behaviour of the matrix is needed, then the fibres must perform at micro-crack level. To achieve this, the fibres must be numerous and must be short with a small diameter. This has to be chosen as such that the workability and placeability are not compromised.

If improvement of the strength and ductility, i.e. macro-level, from a structural behaviour point of view is needed, then longer fibres are needed. These should be long enough that sufficient anchorage length is assured. However, these should be as such that the aspect ratio is not too high thereby negatively influencing the workability [16].

To conclude:
- If increase in bearing capacity is desired, then strong high elastic fibres should be applied.
- Long fibres can also be applied if high ductility and higher bearing capacity are desired after cracking has occurred.
- However, if applying long fibres is not feasible, then using high number of short fibres may increase the ductility at the cost of strength. Short Fibres may also influence the Modulus of Elasticity of the composite.
- If early crack arresting and reducing of early age shrinkage is desired, then one should use a high amount of short flexible fibres keeping in mind the workability of the mortar. Reducing shrinkage reduces the chance of slipping of the joint.

### 4.6 Bridging Action of Fibres

#### 4.6.1 Fibre Bridging

The fibre bridging action of fibres depends on many parameters. The fibres help in dissipating energy through:
- Matrix fracture
- Matrix spalling
- Fibre-matrix interface debonding
- Post-debonding friction between fibre and matrix (pull-out)
- Fibre fraction
- Fibre abrasion
- Plastic deformation of fibres (or yielding)
Studies have shown that the mechanical behaviour of FR Composites is surely influenced by the amount of fibre, orientation of the fibres, and on the pull-out versus load-slip behaviour of the fibres. Moreover, the pull-out is largely determined by the type and the mechanical/geometrical properties of the fibres, on the mechanical properties of the interface between fibre and matrix, on the angle of inclination of the fibre with respect to the direction of the loading and on the mechanical properties of the matrix.

4.6.2 Collaborative Bridging Action between Fibre and Aggregate
Aggregate contribute in bridging the cracks. This phenomenon will act simultaneously with the fibres. Moreover, the fibre-reinforced composite behaviour is a combination of the effects caused by aggregate and fibre bridging, where the aggregate have a very limited affect compared to the fibres.

The figure shows three distinct zones:
- A traction-free zone. This occurs for relatively large crack opening
- Bridging zone. Stress is transferred by fibre pull-out and aggregate bridging
- A zone of micro-cracking and micro-crack growth
These bridging characteristics and the shape of the curve will be affected by the type of fibre, mechanical characteristics (strength, elastic modulus), fibre content, and the aspect ratio of the fibres.

4.7 Hybrid Fibre Reinforced Cementitious Composites

Hybrid Fibre Reinforced Concrete contains two or more types of fibres that in a suitable combination may significantly improve overall properties of the concrete. The combination of fibres can be effective in arresting cracks at both the macro- and micro crack level. By combining fibres in a well-designed manner, the resulting performance can exceed the sum of individual fibre performances. This phenomenon is termed “synergy” [15]. There are mainly three combinations commonly recognized:

- **Hybrids Based on Fibre Constitutive Response:** mechanical properties
  For example, two types of fibres are used with one being stiffer and stronger than the other. Stronger and stiffer fibre provides first crack strength and ultimate strength, while the weaker fibre is relatively flexible and thereby leads to improved toughness (ductility) and strain capacity in the post-crack zone.

- **Hybrids Based on Fibre Dimensions:** aspect ratio
  One fibre is shorter (or smaller) than the other. The smaller fibre bridges micro-cracks and therefore controls their growth and delays crack conjugation resulting in higher toughness and tensile strength of the composite. The larger fibre is then intended to delay propagation of macro-cracks and therefore results in a substantial improvement in the fracture toughness and ductility of the composite.

- **Hybrids Based on Fibre Function:**
  One type of fibre improves the fresh and early age properties such as ease of production and plastic shrinkage, while the other fibre leads to improved mechanical properties.

Non-metallic fibres such as glass, polyester, polypropylene etc. can lead to a reduction of early age cracking, thereby improving the fresh concrete properties. The improvement by these non-metallic fibres is the result of their high aspect ratios and increased fibre strands amount (because of lower density as compared to steel) at a given volume fraction. In addition, because of their lower stiffness, these fibres are particularly effective in controlling the propagation of micro-cracks in the plastic stage of concrete. However, the contribution to post crack behaviour is not that significant as is the case with steel fibres [9].

Some research has been conducted on combinations of steel and polypropylene. It was reported that combining these two fibre types lead to an increase in the ultimate compressive strain. A stronger and stiffer steel fibre improved the ultimate strength, while the more flexible and ductile polypropylene fibre improved toughness and strain capacity in the post crack zone [15].
PART B
EXPERIMENTAL STUDIES
5 Test Set-Up, Joints and Execution

This chapter is particularly of importance in that it clarifies the (pre-) executed works that form the basis of this master’s thesis project. Much of the information in this chapter is described in the literature review of ir. D.C van Keulen [2]. This graduation project is a sub-project of a greater research project.

As mentioned previously, Van Keulen is currently conducting research on high quality mortar joints; the title of his research is “Pre-cast Concrete with High Quality Mortar Joints”. This research aims at answering the following research questions by conducting analytical, numerical and experimental analyses with high quality mortar joints.

The research questions are:

- What is the behaviour of (un-reinforced and profiled) mortar joints with high quality mortar and fibres?
- What is the shear behaviour of mortar joints as opposed to cast-in-situ joints?

With that said, the experiments, mortar joints and their execution will be elaborated in the following subchapters with the goal of clarifying what has been done prior to this thesis project. The information in this chapter will be utilised for this thesis project.

5.1 Experimental Set-Up

5.1.1 Method of Testing

Two series of tests were performed [2]. The first series, consisting of 12 specimens with different joints, was an inventory test series and was used for calibration purposes. Four horizontal joints (i.e. two construction joints and two mortar joints) and eight vertical joints (all mortar joints) were studied. For each joint there were only 1 specimen. In other words, a total of 12 specimens were tested. These specimens were compressed parallel to the direction of the joint until failure. Loading consisted of constant prescribed deformations of increments of 0.1 mm/min.

Samples of the prefab elements and mortar were made to measure the actual compression strength, the splitting tensile strength and the stiffness (young’s Modulus) at the moment of testing the specimens. The samples of the prefab elements consisted of cubes and prisms and were tested according to NEN-EN 12390-3, while the samples of the mortar were tested according to CUR-Recommendation 24.

It was decided by the author and Van Keulen to use the first series for observational purposes in this project. Furthermore, for the second series the number of specimens was increased. However, the number of types of joints was reduced. The joints that were considered for the second series of testing were also part of the first test series.

5.1.2 Experimental Design of Specimens

Two L-shaped pre-cast concrete elements were attached to each other and loaded vertically until failure; see Figure 33 for an example.
The elements of the joints were held together by threaded rods in order to hinder separation of the two elements during transport to the lab and testing. In the case that the elements began to separate from each other, the threaded rods exercised an opposing force, thus keeping the elements together. Furthermore, for some of the elements, pre-stressing was introduced to simulate the normal tension stresses that are present in some structures. For example, in case of the horizontal joints, pre-stressing would simulate the stresses caused by bending (due to wind forces) and weight of the supported elements (such as it is the case with stacked walls). However, for vertical joints, determining the normal tension stresses is dissimilar, since the stresses are not caused by self-weight but by reaction forces, expansion and shrinkage of the surrounding concrete elements.

5.2 Profiled Joints

In high-rise and low-rise buildings, the vertical joints play a role in the transfer of horizontal loads through shear to the foundation. Shear strength is therefore the relevant mechanical properties that was considered for this research project. As mentioned in the first chapter, only profiled mortar joints are considered for this master’s thesis. Figure 34 shows an overview of eight profiled joints (four equal and four shifted profiles), including three containing fibres.
Displayed below are properties and relevant aspects that influence the shear behaviour of the mortar joints.

**Variables:**
- Type of joint: Equal or Shifted Profile
- Pre-stress: 0 N/mm² and 2.0 N/mm²
- Threaded steel rod: M24 or M38
- Fibres: 0 or 30 kg/m³

**Other relevant information concerning the specimens:**
- Width of the joint (varies): 25 - 75 mm
- Total compression diagonals: 3
- Type and quality of the mortar: Cugla Tiksomortel K70
- Type of fibre: Bekaert straight steel wire fibre
- Steel Reinforcement in joint: None
- Concrete quality Pre-cast elements: C53/65

### 5.3 Execution and filling of joint

In this subchapter execution refers to the filling of the vertical juncture between two pre-cast concrete elements with mortar and any other works necessary for the preparation of such juncture. Areas that require attention for the execution of the mortar joint in practice are the configuration or geometry of the joint, the type of mortar (i.e. the material properties) and the method of execution (i.e. grouting, pumping, injecting etc.).
5.3.1 Method of Execution

The following methods of execution are recommended for the vertical joints [24]:

- Grouting
- Pumping

For the first and second series of testing, ‘pumping’ was applied, see Figure 35 and Figure 36. With this method a high level of degree of filling can be achieved. In addition, this method is frequently applied in precast industry to realize vertical joints, thereby making it suitable to incorporate in this research.

Figure 35: Mortar pump

Figure 36 shows the pumped mortar from test. It was necessary to test the pumping of this mortar first before filling the specimens in order to avoid incomplete filling. From this test it was concluded that the FR mortar can be pumped. Furthermore, it was also discovered that in order to clean the pump, the FR mortar in the hose should first be removed by using regular mortar, and subsequently flushed out with water. This is due to the fact that the fibres in the FR mortar had created a cluster when first flushed with water instead of mortar. This was caused by separation of aggregates as a result of the differences in mass between the fine aggregate of the mortar and of the fibres.
5.3.2 Type of Mortar

Trowel mortar with thixotropic\(^4\) properties is recommended for the pumping method \([24]\). To meet this requirement “CUGLA Tiksomortel K70” was used for the elements \([25]\), see Appendix A – CUGLATON TiksoMortar K70. This low-shrinkage mortar is provided by CUGLA BV and is specifically developed for the filling of vertical joints of pre-cast structures, without the use of form-work. The table below shows a brief overview of the properties of this mortar.

<table>
<thead>
<tr>
<th>Table 4: Material properties Tiksomortel K70 ([25])</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Cuglaton TiksoMortel K70</strong></td>
</tr>
<tr>
<td>Mortar Sort</td>
</tr>
<tr>
<td>Maximum size aggregate</td>
</tr>
<tr>
<td>Strength Class</td>
</tr>
<tr>
<td>Characteristic strength after 28 days</td>
</tr>
<tr>
<td>Environment Class</td>
</tr>
<tr>
<td>Swelling</td>
</tr>
<tr>
<td>Average shrinkage</td>
</tr>
</tbody>
</table>

\(^4\) Thixotropy is shear thinning property. Certain gels or fluids that are thick (viscous) under static conditions will flow (become thin, less viscous) when shaken, agitated, or otherwise stressed. They then take a fixed time to return to a more viscous state. In more technical language: some non-Newtonian pseudo-plastic fluids show a time-dependent change in viscosity. (source: Wikipedia)
6 Fibres

This chapter first addresses the fibres by defining basic requirements and boundary conditions for the fibres.

6.1 Requirements and Boundary Conditions

6.1.1 Basic Requirements for Fibres

Certain requirements or specifications regarding fibres are recognized. These requirements are elaborated below, see also [7].

Fibres should be chosen that:

- The aspect ratio is high enough that higher strength and ductility can be achieved. Furthermore, the aspect ratio is of great importance for the workability of the trowel mortar.
- Fibre content is adequate.
- The fibres are corrosion resistant; the fibres should be able to resist the alkaline environment of cementitious materials.

Regarding the fibre properties, there are also requirements. These are summarized below.

- **Tensile strength**
  The fibre must be stronger than the surrounding mortar since the effective load-bearing area at a typical fibre content in a composite is much less than the corresponding area of the matrix.

- **Ductility or Elongation**
  The fibre must be able to withstand strains greater than the cracking strain of the mortar in order to provide significant toughening.

- **Young’s Modulus**
  The fibre must have a high Young’s Modulus relative to that of the mortar in order to carry the load prior to cracking.

- **Elasticity**
  Fibres should possess an elastic behaviour and should not be prone to creep at normal or elevated temperatures. Fibres that behave otherwise suffer stress relaxation when loaded prior to cracking and time-dependent strain after cracking, either of which reduces their reinforcing effectiveness.

Most of the fibres considered for concrete mixes have tensile strengths and elongation characteristics far superior to the normal mortar. Some fibres have creep tendencies at normal or elevated temperatures, like polymeric fibres (i.e. polypropylene and nylon fibres). The most important difference between them from the point of view of reinforcing effectiveness is with respect to Modulus of Elasticity and creep behaviour. Truly elastic, high E-modulus fibres offer greater potential of reinforcing effectiveness than low-modulus elastic fibres or creep–prone fibres where the elastic modulus decreases with loading time. However, realization of the full reinforcing potential depends strongly on the interfacial shear bond between fibre and mortar and whether composite failure ultimately occurs by fibre pullout or fibre breakage [7].
6.1.2 Boundary Conditions
The workability of the FR mortar cannot be neglected as it is relevant for the production of the joint. One important factor of the workability is the fluidity (flow ability) of the mortar. For the trowel mortar it is vital that the flow ability remains intact long enough that the mortar can be pumped. The non-water absorbent fibres reduce this because of the needle like shape and high specific surface. Fibres that absorb water may cause further reduction in mixture fluidity [7].

The workability of the mortar strongly relates to the fibre content. Governing factors for the maximum fibre content are the volume fraction of the paste phase, the maximum aggregate size and the fibre aspect ratio.

Fundamentally, if there are too many fibres and if they are too long or stiff, then the FR mortar cannot be pumped. In other words, the length and diameter (aspect ratio) are relevant for pumping of the mortar. The following dimensions are relevant for the hose of the mortar pump through which the fibres will pass:
- Diameter upper section hose: 50 - 60 mm
- Diameter mouth opening: 20 mm

If the fibres are to be pumped, then it is reasonable to say that the fibres should possess a length no longer than 20 mm based on the diameter of the mouth opening in order to avoid clumping or balling of the fibres. However, it is possible that the fibres will be oriented in the direction of the flow of mortar. So, applying fibres larger than 20 – 25 mm is plausible. Nevertheless, due to the thixotropy of the trowel mortar, it can be assumed that the fibres will not entirely be oriented in the direction of the flow. Therefore, to obtain the best performance of the mortar, it is better to apply fibres with a length of maximum the diameter of the mouth opening. Furthermore, these fibres should be fairly flexible. This allows the fibres to manoeuvre and bend when necessary. In the case of steel fibres, only straight steel wire fibres are considered. Hooked-end or other steel fibre types are, in general too long, and stiff, and can’t be pumped at the given boundary conditions.

6.2 Fibre Properties

6.2.1 Fibre Aspect Ratio
The fibre aspect ratio is the ratio of the length and diameter of the fibre. The fibre aspect ratio can be used to determine suitable fibres based on length and diameter. The aspect ratio is also relevant for the pumping of the mortar.

\[
\text{Fibre aspect ratio: } \frac{L_f}{D_f}
\]

\(L_f\) length of the fibre
\(D_f\) diameter of the fibre

The profiled joints possess a minimum width of 25 mm and a maximum width of 75 mm, see Figure 34). Based on these dimensions, it can be assumed that fibres with a length no longer than 20 - 25 mm should be applied. Otherwise, the advantage of fibres being properly oriented in all directions will be lost. This is also important for the execution, namely pumping, of the joint.

6.2.2 Fibre Content
In general, the maximum fibre amount possible without excessive mixture stiffening and loss of workability is small, often less than 1%, and usually not more than 2% by volume of mixture [7]. In
Subchapter 3.2 it was shown that specimens containing hooked-end steel fibres with contents ranging between 0 and 1.5 percent by volume experienced an increase in strength. Based on the results of this research it can be concluded that mortar joints with higher fibre contents have a relatively higher shear capacity. However, the higher fibre content may compromise the workability of the mortar. It is imperative that an adequate amount of fibres is applied. For the first series of testing a fibre density of 30 kg fibre per cubic metre of mortar was applied.

### 6.3 Type of Fibres

A literature review was conducted on several different types of fibres (metallic and synthetic) that could be used in the fibre reinforced mortar. See Chapter 4. Based on the literature review, two types of fibres were considered for improving the behaviour of the mortar, i.e. straight smooth steel and polyvinyl alcohol (PVA) fibres. The aforementioned requirements and boundary conditions were taken into account. The fibres were chosen based on criterions for strength, stiffness, affordability and the ability to reduce shrinkage in the mortar mix. It was shown that steel fibres provided the best results in terms of strength, stiffness, and affordability. In addition, it was shown that PVA fibres provided the best result in terms of reducing shrinkage due to its high Young's modulus compared to other synthetic fibres. The fibres are displayed in Figure 37. The properties of the fibres are listed in the table below.

<table>
<thead>
<tr>
<th>Property</th>
<th>Steel</th>
<th>PVA RF 1000</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length ( L_f ) [mm]</td>
<td>13</td>
<td>15</td>
</tr>
<tr>
<td>Diameter ( \phi_f ) [mm]</td>
<td>0.2</td>
<td>0.31</td>
</tr>
<tr>
<td>Aspect Ratio ( L_f/\phi_f ) [-]</td>
<td>65</td>
<td>48</td>
</tr>
<tr>
<td>Density ( \rho ) [kg/m³]</td>
<td>7850</td>
<td>1300</td>
</tr>
<tr>
<td>Tensile Strength ( f_t ) [MPa]</td>
<td>2600</td>
<td>1000</td>
</tr>
<tr>
<td>Elastic Modulus ( E ) [GPa]</td>
<td>210</td>
<td>29</td>
</tr>
</tbody>
</table>

![Figure 37: Steel fibres (left) and PVA fibres (right); source: Bekaert & Nycon, respectively](image)
7 Experimental Investigation of Fibre Reinforced Mortar

Prior research has clearly shown that fibres can enhance the performance of cement based composites [11, 12, 26-28]. The addition of steel or polyvinyl alcohol (PVA) fibres has resulted in improvement of different properties, such as lower shrinkage, increase in strength, toughness (ductility) and durability, etc. However, these researches often focussed on concrete as base material and less on mortars, particularly trowel mortars with thixotropic\textsuperscript{5} properties. Moreover, this particular mortar is highly suitable for application in joints of precast concrete structures. Since not much is known about the material behaviour of steel-PVA fibre reinforced mortar, an experimental investigation of the material properties has been carried out in order to answer the following research question:

*How does hybrid steel-PVA fibre reinforced thixotropic mortar behave in terms of shrinkage, compression, tension and flexure?*

This part of the report contains the results of the experimental investigation. The following properties were investigated:

- Workability
- Shrinkage
- Tensile Strength (flexural and splitting)
- Compression Strength

The objective was to investigate the material behaviour with the intention of developing a suitable fibre reinforced mortar (FRM) in terms of tensile (flexural and splitting) and compression strength, toughness (ductility) and shrinkage for application in a precast concrete mortar joint. The goal is to improve the behaviour of the mortar by achieving lower shrinkage, higher yield strength, higher ultimate strength and higher residual strength in the post cracking stage.

This part of the report is structured as follows. First, the experimental approach and program of the different mixtures that were tested in this research are addressed. Subsequently, the production process of the mixtures are expanded on. In this chapter weighing, mixing and casting of the specimens are discussed in detail. Furthermore, the report continues with chapters on the testing methodologies, results and overview of the tested specimens. This was done separately for all three phases of testing. The report concludes with the appendices.

7.1 Experimental Approach and Program

7.1.1 Experimental Approach

The experimental investigation was conducted in three phases of testing:

1. Phase I: Preliminary Tests - determination of shrinkage, maximum flexural and compression strength of twelve mixtures considered in this experiment.
2. Phase II: Secondary Tests - determination of flexural behaviour, compression and splitting tensile strength of the four most promising mixtures of the preliminary tests.

\textsuperscript{5}Thixotropy or pseudo-plasticity is shear thinning property. Certain gels or fluids that are thick (viscous) under static conditions will flow (become thin, less viscous) over time when shearing, agitated, or otherwise stressed. They then take a fixed time to return to a more viscous state. (source: Wikipedia)
3. Phase III: Tertiary Tests - determination of the pump ability of fibre reinforced mortar and the effect of the pump on the material properties.

Preliminary Tests were conducted to acquire a quick overview of the compression strength, maximum flexural strength and shrinkage of the HFRM mixtures. The post cracking behaviour was not investigated in this phase. The investigation of the tensile post-cracking behaviour was conducted only in the second phase following the preliminary tests; the flexural tensile post-cracking behaviour of the four most promising mixtures of the preliminary test was investigated. In this case, the four mixtures with the best results in flexure were considered for further testing. The flexural tests in the second phase were conducted in accordance with NEN-EN 14651 and were carried out in the Stevin laboratory of the faculty of Civil Engineering and Geosciences at the Delft University of Technology.

Lastly, the pumping ability of the four mixtures was examined. In addition, the effect of the pump on the material properties of the four mixtures was also investigated.

In this study, two aspects of the fibres were investigated:

1. The influence of the type of fibre on the material properties; steel versus PVA.
2. The influence of the fibre dosages on the material properties.

7.1.2 Experimental Program

Trial mixtures were prepared to obtain an optimum strength at 7 days along with a good workability. This was done by varying the steel and PVA fibre dosages. In addition, the consistency of the mixture was checked in order to meet the workability requirements as well as requirements set by the pumping ability or passing ability.

The mixture proportions that were considered are presented in the table below.

<table>
<thead>
<tr>
<th>Mix ID</th>
<th>Mixture ID</th>
<th>Volume fraction Steel (%)</th>
<th>Volume fraction PVA (%)</th>
<th>Fibre dosage (kg/m³)</th>
<th>Total fibre dosage (kg/m³)</th>
<th>Fibre dosage per 25 kg mortar [ca. 12 l]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>S1</td>
<td>0.38</td>
<td>0</td>
<td>30.0</td>
<td>30.0</td>
<td>360.0</td>
</tr>
<tr>
<td>3</td>
<td>S2</td>
<td>0.76</td>
<td>0</td>
<td>60.0</td>
<td>60.0</td>
<td>720.0</td>
</tr>
<tr>
<td>O</td>
<td>P1</td>
<td>0.0</td>
<td>0.3</td>
<td>0.0</td>
<td>3.9</td>
<td>3.9</td>
</tr>
<tr>
<td>P</td>
<td>P2</td>
<td>0.0</td>
<td>0.6</td>
<td>0.0</td>
<td>7.8</td>
<td>7.8</td>
</tr>
<tr>
<td>Q</td>
<td>P3</td>
<td>0.0</td>
<td>0.9</td>
<td>0.0</td>
<td>11.7</td>
<td>11.7</td>
</tr>
<tr>
<td>4</td>
<td>S1P1</td>
<td>0.38</td>
<td>0.3</td>
<td>30.0</td>
<td>33.9</td>
<td>360.0</td>
</tr>
<tr>
<td>5</td>
<td>S1P2</td>
<td>0.38</td>
<td>0.6</td>
<td>30.0</td>
<td>37.8</td>
<td>360.0</td>
</tr>
<tr>
<td>6</td>
<td>S1P3</td>
<td>0.38</td>
<td>0.9</td>
<td>30.0</td>
<td>41.7</td>
<td>360.0</td>
</tr>
<tr>
<td>7</td>
<td>S2P1</td>
<td>0.76</td>
<td>0.3</td>
<td>60.0</td>
<td>63.9</td>
<td>720.0</td>
</tr>
<tr>
<td>8</td>
<td>S2P2</td>
<td>0.76</td>
<td>0.6</td>
<td>60.0</td>
<td>67.8</td>
<td>720.0</td>
</tr>
<tr>
<td>9</td>
<td>S2P3</td>
<td>0.76</td>
<td>0.9</td>
<td>60.0</td>
<td>71.7</td>
<td>720.0</td>
</tr>
</tbody>
</table>

C= Control Specimen, S= specimen containing only steel fibres, SP= specimens containing steel and PVA fibres
Note:
The total fibre dosage refers to the total kilograms of fibres per cubic metre of mortar. Therefore, for every mix the actual weight of fibres may vary depending on the volume of the mixture or specimen. For example, if one 25 kg bag of mortar is equal to approximately 12 litres of mortar, then the fibre dosage was recalculated to fit this volume.

7.2 Production Process – weighing, mixing and casting

Uniform fibre distribution is paramount for good quality and good mechanical behaviour. This goes without saying that the mixing method, water content and consistency play an important role in achieving proper fibre distribution. For the tests the water content was set at the maximum amount within the recommended range of 3.8 to 4.2 litres per 25 kg mortar. This was done to compensate for any unknown issues that could’ve occurred as a result of the adverse effects of the fibres on the workability. Moreover, an increase in water content may have been required due to the hydrophilic behaviour of PVA. Though, the water absorption ratio of PVA is less than 1% by weight [29].

First, the weighing, mixing and casting details of the preliminary tests are discussed followed by the secondary and tertiary tests.

Phase I: Preliminary Tests

Weighing details
The fibres, mortar and water dosages were weighed according to the experimental program before mixing took place.

Figure 38: Weighed fibres and weighing of mortar

Mixing Details
In order to ensure a homogenous fibre distribution and to decrease clotting of the steel fibres, the fibres were first mixed with the mortar powder before adding any fluid, see Figure 39. The fibres were gradually poured into the dry mix.
Water was then added to the dry mix to create a mortar solution, see Figure 40.

Figure 40: Adding water to the dry mix for the preliminary tests

_Casting Details_

Fresh mortar was poured in three layers into the steel moulds in order to reduce air entrapment and to realize a homogenous fibre distribution, see Figure 41. In addition, compaction by means of vibration was carried out on a flat table. The following specimens were prepared:

- 40 mm x 40 mm x 160 mm prisms for compression and flexural strength testing
- 40 mm x 40 mm x 160 mm prisms for shrinkage testing

The specimens were de-moulded after 24 hours and placed inside a water tank to cure until the age of testing.
Phase II: Secondary Tests

Mixing Details
In Figure 42 the dry-mixing of mortar powder and fibres and the packaging of the fibre mix for the secondary tests are displayed.

Casting Details
The following specimens were prepared:
- 150 mm x 150 mm x 600 mm prisms to determine the flexural strength and to investigate the flexural tensile post-cracking behaviour
- 150 mm x 150 mm x 150 mm cubes to determine the compression and splitting tensile strength of the mixtures
The specimens were de-moulded after 24 hours and placed inside a fog room to cure until the age of testing. Figure 43 displays the de-moulded prisms and cubes casted in steel moulds for the secondary tests.

Figure 43: Prisms and cubes for secondary tests

Phase III: Tertiary Tests

Weighing and mixing details
A hand mixer was used to mix the fresh mortar as opposed to a mechanical mixer as was shown in subchapter 0, see Figure 44. Furthermore, the same method that was utilized for weighing in the previous phase was also applied in this phase of testing.

Casting Details
The mortar was pumped into the steel moulds as opposed to filling by hand or trowel as was done in the previous phases, see Figure 45 below. No compaction was applied. The following specimens were prepared:

- 40 mm x 40 mm x 160 mm prisms for compression and flexural strength testing

The specimens were de-moulded after 24 hours and placed inside a water tank to cure until the age of testing. The testing methodologies and results are discussed in the following chapters.

Figure 44: Mixing of the mortar with a hand mixer
7.3 Experimental Investigation - Phase I

7.3.1 Testing Methodology
Different test methods are required in order to conduct an experimental investigation of the material properties of the fibre reinforced mortar. These different test methods are discussed in this chapter. Furthermore, in order to further clarify the findings, the specimens were examined and the findings were documented in the form of text and photographs in Subchapters 7.3.2.6.

7.3.1.1 Workability
Prior to casting the specimens, the workability was measured in terms of consistency and spread size of the mixture. The requirement for unreinforced trowel mortar (mortar type < 4 mm) is the following:

- Spread size should range between 140 to 180 mm, determined five minutes after preparing the mortar mix in accordance with CUR-Recommendation 24.
According to the CUR-Recommendation 24, six specimens per mixture are to be tested. A flow table was used in accordance with NEN 3534 to determine the spread size, see Figure 46. Figure 47 shows the filled steel cone. Figure 48 below shows the end result of the workability test.

![Flow Table](source: NEN 3534)

![Filling of the cone (L) and the flow table(R)]

![End result](source: NEN 3534)

### 7.3.1.2 Drying Shrinkage

The drying shrinkage of the mixtures was measured. This was done in accordance with DIN 52450. The requirement for shrinkage is as follows:
- The maximum shrinkage allowed is 1.2 multiplied by the value specified by the manufacturer.
- 24 hour drying shrinkage of TiksoMortar K70; < 0.4 mm/m
- 28 days drying shrinkage of TiksoMortar K70; < 1.2 mm/m

Prisms of 40 x 40 x 160 mm were used for the tests. Two specimens per mixture were tested using a length comparator, see Figure 49.

The shrinkage was measured at the following intervals:
- 24 hours after de-moulding (48 hours after casting)
- 48 hours
- 7 days
- 14 days
- 21 days
- 28 days

Figure 49: Length Comparator

7.3.1.3 Testing Strength of HFRM

The following properties were investigated:
1. Compression Strength in accordance with CUR-recommendation 24
2. Flexural Strength in accordance with CUR-recommendation 24 (ISO 679)

Two specimens were tested each for flexural and compression strength. First, the flexural tests were conducted. Subsequently, the tested specimens were used to determine the compression strength, all in accordance with CUR-Recommendation 24.

Compression Strength

The mortar compression strength \( f_{mk} \) was determined in accordance with CUR-Recommendation (ISO 679 and NEN 5968) at 7 and, subsequently, 28 days after casting. According to the product data sheet of CUGLA, the compression strength of Cuglaton Tiksomortar K70 at 7 days is equal to \( f_{mk}(7) \)
Note that this strength is based on a water dosage of 3.8 litres per 25 kg mortar, as opposed to the 4.2 litres that was applied for all three phases of testing.

In this phase, four prisms per mixture were tested. A total of 4 prisms were casted to test the strength at 7 (2 prisms) and 28 days (2 prisms) for the compressive strength, respectively.

The maximum load was recorded.

**Flexural Strength**

The flexural strength was determined using a three point bending test in accordance with CUR-Recommendation 24 (ISO 679).

According to the product data sheet of CUGLA, the flexural strength of Cuglaton Tiksomortar K70 at 7 days is equal to $f'_{m(7)}=7$ MPa at a water dosage of 3.8 litres per 25 kg mortar. See Appendix A – CUGLATON TiksoMortar K70.

### 7.3.2 Results and Discussion

Countless research has shown that fibres may improve the mechanical properties of cement based composites. Such improvements concern increase in strength, shrinkage reduction, increase in toughness (ductility) etc. [8, 11, 16, 30-32]. Conversely, most researches have been conducted on concrete as base material and less on mortar, particularly thixotropic trowel mortar. The results of the experiments conducted within this graduation project shed more light on the behaviour of this fibre reinforced thixotropic trowel mortar.

#### 7.3.2.1 Flow Spread

The spread size is used as a measurement to determine the workability of cement mixtures. According to CUR-recommendation 24, the requirement for the spread size of traditional trowel mortar ranges between 140 to 180 mm. According to the product data sheet Cuglaton Tiksomortar K70, produced with a water dosage of 3.8 lt/ 25 kg mortar, possesses on average a spread size of 140 mm. On the other hand, it can be observed from Table 7 that the control specimen, produced with a water dosage of 4.2 lt per 25 kg mortar, recorded a slightly larger spread size of 142 mm. As expected, this may be attributed to the higher water dosage, since more water means more flow ability.

In this experiment, all mixtures had a spread size in the range of 127 to 142 mm, see Table 7 below. The spread size of the mixtures containing fibres is lower than the spread size of the traditional trowel mortar (C). Note that this is the spread size at maximum recommended water dosage, namely 4.2 l per 25 kg mortar. It is therefore plausible that the spread size of the fibre reinforced mortar may decrease more for a water dosage of 3.8 l per 25 kg mortar.

<table>
<thead>
<tr>
<th>Mixtures</th>
<th>C</th>
<th>P1</th>
<th>P2</th>
<th>P3</th>
<th>S1</th>
<th>S2</th>
<th>S1P1</th>
<th>S1P2</th>
<th>S1P3</th>
<th>S2P1</th>
<th>S2P2</th>
<th>S2P3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spread Size [mm]</td>
<td>142</td>
<td>139</td>
<td>134</td>
<td>135</td>
<td>136</td>
<td>134</td>
<td>127</td>
<td>132</td>
<td>131</td>
<td>134</td>
<td>130</td>
<td></td>
</tr>
</tbody>
</table>

The spread size can also be expressed in terms of a fibre reinforcement index FRI, which is equal to the volume fraction multiplied by the fibre aspect ratio. Groth & Nemegeer [33] used it to describe...
the effect of contents and type of steel fibres on the slump flow, which can be considered the same as the spread size. The fibre reinforcement index was calculated as follows:

\[ FRI = \frac{v_f}{\phi_f} \cdot \frac{L_f}{\phi_f} \]  

(7.1)

Where

- \( v_f \) fibre volume fraction [-]
- \( L_f \) fibre length [mm]
- \( \phi_f \) fibre diameter [mm]

The fibre reinforcement index was determined for the steel and PVA fibres. In order to compare the different mixtures and to combine the effect of both fibres for the hybrid mixtures, the individual FRI values for steel and PVA were summed up to form a total FRI. The total FRI values \((S+P)\) of the different mixtures are listed in Table 8. The mixtures are sorted in increasing order of total FRI.

Figure 50 displays the spread size versus the total FRI. It can be clearly observed that the spread size decreases with increasing FRI. There is an apparent relationship between the spread size and the total FRI. Mixtures S2 and S2P2 are likely outliers. When comparing the mixtures individually, some interesting discoveries can be made. In some cases, it seems as if the differences between the control specimen and the mixtures containing an individual fibre type can be summed up to determine the workability of the hybrid mixtures. For example, the difference between C and P1 or C and S1 is 3 mm or 6 mm, respectively. The difference between C and S1P1 is 9 mm \((3+6)\). Another example is the difference between S1 and S2, which is 2 mm. This difference is similar to the differences between S1P1 and S2P1 or S1P3 and S2P3; 133 – 131 mm or 132 – 130 mm, respectively. In the latter cases, the PVA fibre dosage remained the same as opposed to the steel fibre dosage. Whether this is coincidence or not, it could not be determined as more tests would be required. Note that these values are based on only one measurement and therefore the scatter is unknown.

### Table 8: Fibre reinforcement index versus the spread size

<table>
<thead>
<tr>
<th>Mixtures</th>
<th>Fibre Volume Fraction [%]</th>
<th>Fibre Reinforcement Index FRI [%]</th>
<th>Spread Size [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Steel</td>
<td>PVA</td>
<td>Steel</td>
</tr>
<tr>
<td>C</td>
<td>0</td>
<td>0.00</td>
<td>0.0</td>
</tr>
<tr>
<td>P1</td>
<td>0</td>
<td>0.30</td>
<td>0.0</td>
</tr>
<tr>
<td>S1</td>
<td>0.38</td>
<td>0.00</td>
<td>24.7</td>
</tr>
<tr>
<td>P2</td>
<td>0</td>
<td>0.60</td>
<td>0.0</td>
</tr>
<tr>
<td>S1P1</td>
<td>0.38</td>
<td>0.30</td>
<td>24.7</td>
</tr>
<tr>
<td>P3</td>
<td>0</td>
<td>0.90</td>
<td>0.0</td>
</tr>
<tr>
<td>S2</td>
<td>0.76</td>
<td>0.00</td>
<td>49.4</td>
</tr>
<tr>
<td>S1P2</td>
<td>0.38</td>
<td>0.60</td>
<td>24.7</td>
</tr>
<tr>
<td>S2P1</td>
<td>0.76</td>
<td>0.30</td>
<td>49.4</td>
</tr>
<tr>
<td>S1P3</td>
<td>0.38</td>
<td>0.90</td>
<td>24.7</td>
</tr>
<tr>
<td>S2P2</td>
<td>0.76</td>
<td>0.60</td>
<td>49.4</td>
</tr>
<tr>
<td>S2P3</td>
<td>0.76</td>
<td>0.90</td>
<td>49.4</td>
</tr>
</tbody>
</table>
It is plausible that in addition to the fibre volume fraction and aspect ratio, the flexural stiffness of the fibres may influence the flow of the fibre reinforced mortar in the fresh state. Therefore, when the fibre reinforcement index is extended with a factor that expresses the flexural stiffness of the fibres, similar relations can be found, see Figure 51. The extended fibre reinforcement index was calculated as follows:

$$ FRI_{ex} = v_f \cdot \frac{L_f}{\phi_f} \cdot EI $$  \hspace{1cm} (7.2)

Where

$$ EI $$ stiffness of the fibres [Nmm$^2$]

The stiffness of the fibres were calculated as follows:

$$ Steel \rightarrow EI = 210000 \cdot \frac{\pi}{64} \cdot \phi_f^4 \approx 16.5 \text{ Nmm}^2 $$ \hspace{1cm} (7.3)

$$ PVA \rightarrow EI = 29000 \cdot \frac{\pi}{64} \cdot \phi_f^4 \approx 13.4 \text{ Nmm}^2 $$ \hspace{1cm} (7.4)

Furthermore, a total extended fibre reinforcement index was calculated for each mixture. Figure 51 displays the spread size versus the total extended fibre reinforcement index. It can be observed that the shape of the curve changes compared to the curve in Figure 50. It can also be observed that the position of some of the mixtures on the curve have switched. This applies to S1 and P2, S1P1 and P3, S2 and S1P2 and lastly S2P1 and S1P3. Table 8 and Table 9 also display the position of the mixtures in increasing order of total FRI clearly.
Table 9: Extended fibre reinforcement index versus the spread size

<table>
<thead>
<tr>
<th>Mixtures</th>
<th>Fibre Volume Fraction [-]</th>
<th>Fibre Reinforcement Index FRI [Nmm$^2$]</th>
<th>Spread Size [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Steel</td>
<td>PVA</td>
<td>Steel</td>
</tr>
<tr>
<td>C</td>
<td>0.00</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>P1</td>
<td>0.30</td>
<td>0.0</td>
<td>190.8</td>
</tr>
<tr>
<td>P2</td>
<td>0.60</td>
<td>0.0</td>
<td>381.7</td>
</tr>
<tr>
<td>S1</td>
<td>0.38</td>
<td>0.0</td>
<td>407.4</td>
</tr>
<tr>
<td>P3</td>
<td>0.90</td>
<td>0.0</td>
<td>572.5</td>
</tr>
<tr>
<td>S1P1</td>
<td>0.38</td>
<td>0.30</td>
<td>407.4</td>
</tr>
<tr>
<td>S1P2</td>
<td>0.38</td>
<td>0.60</td>
<td>407.4</td>
</tr>
<tr>
<td>S2</td>
<td>0.76</td>
<td>0.0</td>
<td>814.8</td>
</tr>
<tr>
<td>S1P3</td>
<td>0.38</td>
<td>0.90</td>
<td>407.4</td>
</tr>
<tr>
<td>S2P1</td>
<td>0.76</td>
<td>0.30</td>
<td>814.8</td>
</tr>
<tr>
<td>S2P2</td>
<td>0.76</td>
<td>0.60</td>
<td>814.8</td>
</tr>
<tr>
<td>S2P3</td>
<td>0.76</td>
<td>0.90</td>
<td>814.8</td>
</tr>
</tbody>
</table>

![Figure 51: Spread size versus total extended fibre reinforcement index](image)

In both cases, the spread size appears to converge to a minimum value with increasing FRI, which may imply that there is a minimum value after which the spread size could be rendered ineffective to measure the workability. Certainly, there are also other factors that may determine the maximum FRI that is allowed, such as the pumping or passing ability.

Furthermore, from the workability tests it was observed that the mixtures containing higher amounts of fibres tended to experience a quicker hardening rate in the fresh state. This caused a faster loss of workability. This was particularly evident for mixture S2P3, while the other mixtures experienced this phenomenon less. Therefore, pumping with this mixture for example could prove to be problematic.

### 7.3.2.2 Drying Shrinkage

Two different types of shrinkage are distinguished; plastic shrinkage and drying shrinkage. The plastic shrinkage is defined as the shrinkage that occurs in the first 24 hours after casting. Freshly casted mortar will experience, particularly in a non-ideal environment, higher drying rates on the surface if
not covered. This shrinkage in the plastic phase may lead to cracks. Unfortunately, the plastic shrinkage could not be investigated, while this shrinkage phenomenon is most relevant for the PVA fibres. Moreover, literature states that these fibres play an important role in the early age shrinkage of the cement mixtures and have less influence on the drying shrinkage [12]. Indeed, from the results it can be concluded that the PVA fibres do not play a significant role in the drying shrinkage, see Figure 52.

To measure the drying shrinkage, measurements of two specimens per mixture were taken at 1, 2, 5, 7, 14, 21, and 28 days. The measurements follow after the so-called zero measurement, which takes place 24 hours after casting. The following observations can be made about the drying shrinkage. At 28 days, the PVA fibre mixtures appear to have undergone a larger shrinkage compared to the mixtures containing steel fibres, see Figure 53. In fact, the shrinkage of the PVA fibre mixtures (approximately the same for both P1 and P3) is 8.5% larger than S2P1 and 14.5% larger than S1P1, see Figure 53. This may imply some effect of the steel fibres on the drying shrinkage, since these mixtures recorded a lower drying shrinkage compared to the mixtures containing PVA fibres. This difference may be attributed to the high stiffness of the steel fibres, which is in accordance with [12]. Furthermore, the difference in drying shrinkage between the different mixtures containing fibres is less than 0.2 mm/m after 28 days.

Not much else can be said about the effect of the steel fibres, except that there appears to be no mutual relation between the steel fibre dosage and the shrinkage of mixtures containing steel fibres. For example, it can be observed that the S2P1 recorded a larger shrinkage than S1P1, despite S2P1 containing two times as much steel fibres. However, a lower shrinkage was found for S2; almost as low as for S1P1. Ultimately, in order to test if the fibres have really contributed to the drying shrinkage at a water dosage of 4.2 lt/25 kg mortar, measurements of the traditional mortar using the same water dosage is required. Unfortunately, these measurements were not conducted.

Lastly, according to the product data sheet the traditional mortar undergoes a maximum shrinkage of 1.2 mm/m at a water dosage of 3.8 lt/25 kg mortar, see Appendix A – CUGLATON TiksoMortar K70. However, from the graph it can be observed that the lowest recorded shrinkage is still roughly 21% larger than the reference, despite the presence of fibres in the mixture. Note that these mixtures were treated with a larger water dosage of 4.2 lt/25 kg mortar; a difference of 0.4 lt/25 kg mortar. This large difference in shrinkage as a result of the small difference in water dosage may indeed strongly imply that the water dosage has the largest effect on the drying shrinkage. Therefore, in terms of shrinkage, the water dosage should be kept as low as is feasible.
7.3.2.3 Compression Strength

Four measurements were executed for the compression strength at ages 7 and 28 days, respectively. The averages of these results are displayed in Figure 54 and in Appendix C – Results of Preliminary Tests. The following could be observed. Except in the case of S1, all specimens containing steel fibres lead to a marginal improvement in compression strength at 7 days. The compression strength appears to have not been positively influenced by the presence of PVA fibres. Furthermore,
specimens containing 60 kg/m$^3$ of steel fibres (S2) provided a greater increase in compression strength compared to 30 kg/m$^3$ (S1), though marginal. At 28 days, a marginal improvement in compression strength was recorded for all specimens containing steel fibres compared to the control specimen. Again it can be observed that a higher compression strengths was recorded for the specimens containing higher dosage of steel fibres. Amongst the mixtures containing steel fibres, the highest compression strength was recorded for S2P1 while the lowest strength was recorded for S1P1. Needless to say, the mixtures containing 60 kg/m$^3$ of steel fibres are the clear victors regarding the compression strength. Furthermore, no clear relationship could be observed between the fibre dosages and the compression strength.

![Figure 54: Average compression strength at 7 and 28 days](image)

### 7.3.2.4 Flexural Strength

From the results it can be observed that all mixtures containing fibres, except P3, increased in flexural strength. The increase in strength is significant for the mixtures containing steel fibres and for mixture P1. In addition, the hybrid mixtures containing the lowest dosage of PVA fibres (SxP1) appeared to gain a higher flexural strength compared to the hybrid mixtures containing higher PVA fibre dosages. Also, when S1 and S2 are compared to S1Px and S2Px, respectively, it seems that there’s no increase in strength with the addition of PVA fibres. Interestingly, a lower increase in strength was recorded for S2 when compared to S1. Moreover, a lower average flexural strength was measured for S2, see Figure 55. Also see Appendix C – Results of Preliminary Tests. This can be explained by the fact that a much lower flexural strength was recorded for the second specimen compared to the first. This is further clarified in subchapter 0. It was discovered that the second prism boasted a subpar fibre distribution. This illustrates the importance of fibre distribution and the affect it may have on the mechanical properties of mortar. Moreover, it is known that a
homogeneous fibre distribution and orientation of fibres may influence the fresh and hardened concrete properties.

Conversely, at 28 days, different discoveries can be made. Only mixtures containing S2 fibres displayed an improvement of 11% or more as opposed to the other mixtures when compared to the control specimen. Additionally, it can be observed that the PVA fibres of the hybrid mixtures did not further increase the flexural tensile strength. S2P3 is the only exception in this case. Again, the results show that specimens containing 60 kg/m³ of fibres (S2) provided higher flexural strength. The increase in strength in terms of percentage is almost twice as much for S2 as for S1. Furthermore, no clear relationship between the flexural strength and fibre dosages could be observed.

7.3.2.5 *Flexure versus Compression*

Ratios between the flexural and compression strength for the different mixtures were computed to determine if there’s a relation, see Table 10. It can be observed that for mixtures S2 to S2P2 and P1 the ratios are similar at 7 days. Figure 56 displays the ratio against the fibre reinforcement index of the mixtures. The graph shows that a good relation between the ratio and FRI can be defined. A somewhat linear (or constant) relation can be observed. A much higher ratio was computed for S1, which is likely related to the high flexural strength that was recorded. Lower ratios were computed for S2P2, P2 and P3. S1 and P3 are potential outliers.

Interestingly, the ratios at 28 days did not follow the same trend as with the ratios at 7 days. It can be observed that the ratios decreased with increasing FRI, see Figure 56.

Additionally, it appears that the ratios have decreased over time. The reason behind this could not be established. Furthermore, the ratio of the control specimen remained relatively the same over time. This may imply that the ratio between the flexural strength and compression strength of the
unreinforced mortar can be altered and that the development of the flexural as well as the compression strength over time changes by the presence of fibres.
### Table 10: Ratio Flexural to Compression Strength

<table>
<thead>
<tr>
<th>Mixtures</th>
<th>C</th>
<th>S1</th>
<th>S2</th>
<th>S1P1</th>
<th>S1P2</th>
<th>S1P3</th>
<th>S2P1</th>
<th>S2P2</th>
<th>S2P3</th>
<th>P1</th>
<th>P2</th>
<th>P3</th>
</tr>
</thead>
<tbody>
<tr>
<td>FRI</td>
<td>0.00</td>
<td>24.70</td>
<td>49.40</td>
<td>39.22</td>
<td>53.73</td>
<td>68.25</td>
<td>63.92</td>
<td>78.43</td>
<td>92.95</td>
<td>14.52</td>
<td>29.03</td>
<td>43.55</td>
</tr>
<tr>
<td>7 days</td>
<td>Flexural</td>
<td>10.08</td>
<td>12.87</td>
<td>12.39</td>
<td>12.31</td>
<td>11.53</td>
<td>11.89</td>
<td>12.67</td>
<td>12.56</td>
<td>11.51</td>
<td>11.58</td>
<td>10.21</td>
</tr>
<tr>
<td>Compression</td>
<td>66.00</td>
<td>65.60</td>
<td>72.33</td>
<td>70.18</td>
<td>67.28</td>
<td>71.13</td>
<td>72.48</td>
<td>72.82</td>
<td>71.45</td>
<td>65.28</td>
<td>64.83</td>
<td>65.78</td>
</tr>
<tr>
<td>Ratio</td>
<td>0.153</td>
<td>0.196</td>
<td>0.171</td>
<td>0.175</td>
<td>0.171</td>
<td>0.167</td>
<td>0.175</td>
<td>0.172</td>
<td>0.161</td>
<td>0.177</td>
<td>0.157</td>
<td>0.128</td>
</tr>
<tr>
<td>28 days</td>
<td>Flexural</td>
<td>12.04</td>
<td>13.05</td>
<td>13.82</td>
<td>13.27</td>
<td>12.88</td>
<td>13.20</td>
<td>13.60</td>
<td>13.39</td>
<td>14.04</td>
<td>12.71</td>
<td>13.02</td>
</tr>
<tr>
<td>Compression</td>
<td>80.17</td>
<td>81.45</td>
<td>86.70</td>
<td>80.00</td>
<td>82.73</td>
<td>82.88</td>
<td>87.18</td>
<td>87.02</td>
<td>87.00</td>
<td>75.15</td>
<td>76.98</td>
<td>78.00</td>
</tr>
<tr>
<td>Ratio</td>
<td>0.150</td>
<td>0.160</td>
<td>0.159</td>
<td>0.166</td>
<td>0.156</td>
<td>0.159</td>
<td>0.156</td>
<td>0.154</td>
<td>0.161</td>
<td>0.169</td>
<td>0.169</td>
<td>0.159</td>
</tr>
</tbody>
</table>

**Figure 56**: Ratio flexure to compression versus FRI at 7 days (left) and 28 days (right)
7.3.2.6 Overview of Tested Specimens

Some of the specimens tested at 7 days were observed and important facts were documented. The intention was to use these observations to help further clarify the results that were discussed in the previous chapter.

**Specimen S1**

Figure 57 below displays the cracked surface of the specimens. From the image two important aspects can be observed, the fibre distribution and the direction of the fibres. In general, the fibres appear to be protruding sufficiently from the cracked surface, which suggests that the fibre orientation may have had a positive influence on the material properties. Secondly, the fibre distribution appears to be similar for both specimens. Indeed, the graph displaying the flexural strength for the different mixtures shows a small standard deviation for S1 at 7 days (see Figure 158), suggesting that the fibre distribution and perhaps orientation was sufficient to have had a positive and relatively similar effect in both specimens.

![Specimen A](image)

![Specimen B](image)

**Figure 57: Cracked cross-section of specimen S1**

**Specimen S2**

Again, the fibre orientation and the fibre distribution can be clearly observed in Figure 58. Interestingly, the fibre distribution appears to be subpar in specimen B when compared to specimen A. In fact, the consequence of this inferior fibre distribution can be clearly observed in the graph displaying the flexural strength, see Figure 158. S2 (at 7 days) shows a much larger standard deviation than the other mixtures. This specimen recorded a much lower flexural strength than the counterpart. Once more, it illuminates the effect that fibre distribution may have on the flexural strength. Fortunately, a subpar fibre distribution was only observed for this specimen, which may suggest that the execution or casting procedure of the specimens is sufficient.
Specimen S1P1
From the image a relatively homogenous fibre distribution can be observed. The PVA fibres are also visible. In addition, the fibre orientation appears to be adequate for both mixtures. Indeed, the fibres appear to protrude sufficiently from the cracked surface for both specimens.

Specimen S1P3
Figure 60 displays an interesting situation regarding the fibre orientation. It can be observed that the steel fibres appear to protrude from the cracked surface, whereas many of the PVA fibres appear to lay relatively flat on the cracked surface. This may have resulted in a lower flexural strength than expected. The graph shows that S1P3 recorded a lower flexural strength than S1P1, see Figure 158. This may have indeed been attributed to the inferior fibre orientation. Conversely, the fibre distribution appears to be even.
Specimen S2P1
From the image below it can be observed that the fibres appear to protrude sufficiently from the cracked surface. Also, the fibre distribution appears to be fairly even, particularly for the steel fibres.

Specimen S2P3
Figure 62 displays interesting findings concerning the fibre distribution and fibre orientation. It is noticeable that the fibre orientation in specimen B appears to be largely angled to one direction, i.e. to the right. This may have had an adverse affect on the flexural strength. In addition, the steel fibre distribution appears to be largely unfavourable in specimen B. Furthermore, specimen A appears to have contained an air pocket, which may have adversely affected the flexural strength significantly. In both specimens, it seems as if the PVA fibres are overwhelming the steel fibres. In terms of strength, this might not be favourable and may have also significantly affected the flexural strength negatively. It can be seen from the bar graph that these specimens recorded the lowest flexural strength amongst the mixtures containing steel fibres.
7.3.3 Concluding Remarks

**Spread Size**

The results show that there might be some relationship between the fibre reinforcement index and the spread size. The spread size appears to converge to a minimum value regardless of FRI. In some cases it was possible to sum up the differences of the mixtures with individual fibres to determine the spread size of the hybrid counterparts.

**Shrinkage**

No significant distinction can be made concerning the shrinkage of mixtures containing steel fibres. However, there is a notable difference between the mixtures containing steel fibres and those containing only PVA fibres. This may be largely attributed to the considerable difference in stiffness between the steel and PVA fibres. Furthermore, the water dosage may have had the largest effect on the drying shrinkage.

**Compression Strength**

The findings evidently display an improvement in compression strength as a result of the fibres. The limit appears to be 10% for the fibre dosages applied. It was also observed that a higher steel fibre dosage resulted in higher compression strength. Interestingly, all specimens containing S2 appeared to consistently increase in strength by approximately 10% and 20% from 7 to 28 days, respectively.

**Flexural Strength**
Again, the results provide compelling evidence that the flexural tensile strength may be significantly improved by adding fibres to the mix. Most prominent mixtures were S1, S2, S1P1, S2P1 and S2P2. Increases in strength higher than 20% after 7 days were recorded for these mixtures. However, no clear relationship between the fibre dosages and the flexural strength could be observed.

Furthermore, there seems to be a relation between the flexural and compression strength. This was expressed in ratios that showed a somewhat linear relation between the ratios and the fibre reinforcement index at 7 days. A clear relationship between the ratios and the FRI at 28 days could not be established. In addition, the ratios appeared to decrease with increasing FRI. Furthermore, the ratios decreased in time, except in the case of the unreinforced mortar. This may imply that the fibres alter the relation between the flexural and compression strength and the development of the strengths.

7.4 Experimental Investigation - Phase II

7.4.1 Testing Methodology

7.4.1.1 Compression

The compression strength was determined in accordance with NEN-EN 12390-3 at 8 days after casting.

According to the product data sheet of CUGLA, the characteristic cube strength of Tiksomortar K70 at 28 days is equal to $f_{mk(28)} = 81$ MPa. See Appendix A – CUGLATON TiksoMortar K70. Again, it is worth noting that this is the strength based on a water dosage of 3.8 litres per 25 kg mortar, as opposed to 4.2 litres that was also applied in this phase of testing.

Three cubes per mixture were tested at a loading rate of 13.5 kN/s. The maximum load was recorded.

The compression strength is given by the following expression:

\[ f_{cc} = \frac{F}{A} \text{MPa} \]  \hspace{1cm} (7.5)

7.4.1.2 Splitting Tension

The splitting tensile strength was determined in accordance with NEN-EN 12390-6 at 8 days after casting.

The characteristic splitting tensile strength of TiksoMortar K70 is currently unknown.

Three cubes per mixture were tested at a loading rate of 1.35 kN/s. The maximum load was recorded.

The splitting tensile strength is given by the following expression:

\[ f_{fct, spl} = \frac{2F}{\pi \times L \times d} \text{MPa} \]  \hspace{1cm} (7.6)

Where

F     Load corresponding to the tensile strength
L     height of the cube (150 mm)
7.4.1.3 Flexure

The flexural behaviour was investigated by means of a three point bending test in accordance with NEN-EN 14651.

Parameters

The specimens were tested on a displacement controlled testing machine with measuring gauges (LVDT’s) placed on both sides of the specimen; 2 LVDT’s for the vertical displacement and the CMOD respectively. Figure 64 depicts an experimental set-up of the test. This was necessary in order to obtain a full picture of the flexural behaviour by means of load-displacement curves, which can be used for analysis and data interpretation. The following parameters were measured; the force (F), the vertical displacement (δ), crack mouth opening displacement (CMOD) and the displacement (s) of the cylinder applying the force.

The results of the tests were used for calculation of various parameters, such as first crack and ultimate flexural strength and residual strength (ductility). From these parameters a constitutive model could be developed based on a stress-crack width/strain method for flexural behaviour.
Limit of Proportionality (LOP)

According to NEN-EN 14651 LOP is the stress at the tip of the notch which is assumed to act in an uncracked mid-span section, with linear stress distribution of a prism subjected to the centre-point load F. Figure 65 displays a characteristic curve of the three point bending test.

This can be calculated using the following equations:

\[
f = \frac{1}{4} \frac{F_i \cdot L_d}{b h_{lig}^2} = \frac{3 \cdot F_i \cdot L_d}{2 b h_{lig}^2}
\]

(7.7)

The residual flexural tensile strength \( f_{R_i} \) can be calculated with the following expression:
\[
    f_{R,j} = \frac{1}{4} \frac{F_j \cdot L_d}{b h_{ig}^2} = \frac{3 \cdot F_j \cdot L_d}{2 b h_{ig}^2}
\]  

(7.8)

Where

- \( f \) the LOP [MPa]
- \( f_{R,j} \) the stress corresponding with CMOD\(_j\) (j=1, 2, 3, 4) [MPa]
- \( F_t \) the load corresponding to LOP [N]
- \( F_j \) the load corresponding with CMOD\(_j\) (j=1, 2, 3, 4) [N]
- \( L_d \) the span length [mm]
- \( b \) the width of the specimen [mm]
- \( h_{ig} \) distance between the tip of the notch and the top of the specimen [mm]

The following values were set for CMOD; CMOD\(_1\) = 0.5 mm, CMOD\(_2\) = 1 mm, CMOD\(_3\) = 1.5 mm, CMOD\(_4\) = 2 mm. These values deviate from the standard values in the norm. The reason thereof is that the LVDT’s that were used to measure the CMOD measured only up to approximately 2 mm as opposed to 5 mm in the norm.

**Notching**

Before testing commenced, all the specimens were notched by means of wet sawing. See Appendix G – Photos. The height of the notch was set at \( a = 25 \) mm.

![Notching and prepared prism with fixtures and LVDT's](image)

*Figure 66: Notching and prepared prism with fixtures and LVDT's*

**Displacement Rates**

Three prisms per mixture were tested at the following displacement rates:

- 0.0004 mm/s (0.024 mm/min); before the ultimate strength was achieved.
- 0.001 mm/s (0.06 mm/min); rate was increased after the ultimate flexural strength was achieved.

These displacement rates proved to be adequate for the flexural tests and are based on literature [26]. Note that these are not the crack opening displacement rates, but rather the vertical displacement rates of the cylinder applying the force.
7.4.2 Results and Discussion

7.4.2.1 Compression Strength
As mentioned previously the compression strength was determined using cubes. Three measurements were taken at an age of 8 days. The results of these measurements are presented in the graph below. The graph shows that higher compression strengths were recorded for mixtures S2 and S2P1 containing 60 kg/m$^3$ of steel fibres, though marginal compared to S1P1. It seems as if both PVA and steel fibres may have had a minimal effect in increasing the compression strength.

![Compression Strength Secondary Tests](image_url)

Figure 67: Compression Strength of mixtures

The table below displays the size factors between the compression strength of mixtures S1, S2, S1P1 and S2P1 of the preliminary and secondary tests. Interestingly, the ratios of the different mixtures are in the same order, implying a level of reliability of the measurements. This may also suggest that the size effect is independent of the fibre dosage and is only dependent on the dimensions of the specimens that are being compared. Furthermore, the higher ratio of mixture S1 could advocate that the cube of mixture S1 performed better in comparison to other mixtures or that the prism performed less compared to the specimens of the other mixtures.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>S1</th>
<th>S2</th>
<th>S1P1</th>
<th>S2P1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression [N/mm²]</td>
<td>Average Cubes</td>
<td>59.64</td>
<td>62.37</td>
<td>61.91</td>
</tr>
<tr>
<td></td>
<td>Average Prisms</td>
<td>65.60</td>
<td>72.33</td>
<td>70.18</td>
</tr>
<tr>
<td>Ratio</td>
<td>0.91</td>
<td>0.86</td>
<td>0.88</td>
<td>0.87</td>
</tr>
</tbody>
</table>
### 7.4.2.2 Splitting Tensile Strength

From Figure 68 it can be observed that higher splitting tensile strengths were recorded for mixtures S2 and S2P1 compared to S1 and S1P1. Furthermore, the standard deviations of the four mixtures are relatively small. Table 12 shows that the PVA fibres may have had a significant effect on the splitting tensile strength. In fact, a higher increase in strength of approximately 9% was recorded for S2P1 as opposed to S2 when compared to S1. Also, a higher splitting tensile strength was recorded for S2P1 compared to S2. An increase of 7.6% was recorded. However, the increase in strength was less for S1P1 compared to S1; to be exact it was only 2.73%. It is likely that there was a dissimilarity in the amount of fibres crossing the cracked plane of the different specimens, which may have caused a difference in increase in strength. In fact, it is known that the splitting tensile strength is largely governed by the amount of fibres that crosses the cracked plane. The difference in increase in strength may also be correlated to the increase in steel fibre dosage. Moreover, there may have been some collaboration between the steel and PVA fibres that increased as the steel fibre dosage increased. In other words, the higher steel fibre dosage may have lead to a better effect of the PVA fibres in terms of splitting tensile strength.

Another possible explanation for this increase in strength is the enhanced adhesive bond of the PVA fibres. PVA fibres possess a hydrophilic behaviour, which may result in stronger adhesive bonds between matrix and fibre compared to the bond between matrix and steel fibres. It is stated that the combination of bond strength, high tenacity and modulus provides PVA reinforced mortar with better tensile and flexural behaviour [34]. This effect, combined with a possible collaboration with the higher steel fibre dosage, could have caused this significant increase in strength for mixture S2P1.
Table 12: Increase in Splitting Tensile strength compared to S1

<table>
<thead>
<tr>
<th>Mixture</th>
<th>S1</th>
<th>S2</th>
<th>S1P1</th>
<th>S2P1</th>
<th>Note</th>
</tr>
</thead>
</table>
| [B] Average | 4.88 | 5.43 | 5.01 | 5.85 | Percentage Increase 0.00 0.11 0.03 0.20 e.g: [B] Sx(Px)/S1 - 1

7.4.2.3 Flexural Strength

Maximum Flexural Strength (LOP)

Figure 69 displays the results of the flexural tests. Interestingly, S1P1 had a higher flexural strength compared to S2 despite S2 containing twice as much steel fibres. In terms of increase in maximum flexural strength, there appears to be only a small difference between S1P1 and S2P1. However, when S2 is compared to S2P1 an increase of 5% can be observed. There’s no clear evidence as to what caused these differences. The PVA fibres may have had an effect, since a higher flexural strength was recorded for S1P1 compared to S1. The same goes for S2P1 and S2. Nevertheless, the increase in strength is small. Other factors such as fibre orientation and distribution may have contributed to these differences. Lastly, there may have been some kind of collaboration between the PVA and steel fibres as suggested in the previous subchapter.

Flexural Strength Secondary Tests

<table>
<thead>
<tr>
<th>Mixtures</th>
<th>Flexural Strength N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>5.12 N/mm²</td>
</tr>
<tr>
<td>S2</td>
<td>5.83 N/mm²</td>
</tr>
<tr>
<td>S1P1</td>
<td>6.05 N/mm²</td>
</tr>
<tr>
<td>S2P1</td>
<td>6.10 N/mm²</td>
</tr>
</tbody>
</table>

Figure 69: Maximum Flexural Strength of the Mixtures

Table 13: Flexural Strength of Mixtures

<table>
<thead>
<tr>
<th>Mixtures</th>
<th>S1</th>
<th>S2</th>
<th>S1P1</th>
<th>S2P1</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ft [kN]</td>
<td>16.00</td>
<td>18.22</td>
<td>18.88</td>
<td>19.07</td>
<td>Percentage Increase 0% 14% 18% 19% e.g: [A] Sx(Px)/S1 - 1</td>
</tr>
<tr>
<td>f [N/mm²]</td>
<td>5.12</td>
<td>5.83</td>
<td>6.04</td>
<td>6.10</td>
<td></td>
</tr>
</tbody>
</table>
Residual Flexural Strength

Figure 70 and Figure 71 display the residual flexural strengths corresponding to four different CMOD values of the four mixtures. It can be clearly observed that the residual flexural strengths for S2 and S2P1 are much higher than S1 and S1P1. Small differences can be observed between S2 and S2P1. Furthermore, S1P1 had higher residual strengths than S1. This increase in strength may be attributed to the PVA fibres. Literature states that the fibres may provide additional ductility to mortar or concrete [34]. Also see Appendix B – Kuralon PVA. However, no contribution could be observed in mixture S2P1 when compared to S2, despite the PVA fibres’ contribution to S1P1 with respect to S1. It can also be observed that the scatter in mixture S2P1 was large. This coincides with the findings concerning fibre orientation and distribution in Chapter 7.4.2.7. Indeed, the fibre orientation and distribution were different for all three specimens, which may have resulted in a larger scatter. Furthermore, it must be noted that the flexural strength of S2 is based on only one measurement as the results of the other two specimens were unusable. Therefore, the average value could be lower, since the scatter is unknown for this mixture. Assuming that the average value becomes lower, the difference between S2 and S2P1 would decrease and therefore the effect of the PVA fibres could become even clearer or vaguer.

Figure 70: Residual Flexural Strength of Mixtures – Mixtures on x-axis
## Residual Flexural Strength

![Graph of Residual Flexural Strength](image)

**Mixtures**

Figure 71: Residual Flexural Strength of Mixtures - CMOD on x-axis

### Table 14: Residual Flexural Strength

<table>
<thead>
<tr>
<th></th>
<th>S1</th>
<th>S2</th>
<th>S1P1</th>
<th>S2P1</th>
</tr>
</thead>
<tbody>
<tr>
<td>fr [N/mm²]</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CMOD1=0.5 mm</td>
<td>3.09</td>
<td>4.57</td>
<td>2.76</td>
<td>4.34</td>
</tr>
<tr>
<td>CMOD2=1 mm</td>
<td>2.50</td>
<td>4.63</td>
<td>2.81</td>
<td>4.43</td>
</tr>
<tr>
<td>CMOD3=1.5 mm</td>
<td>2.29</td>
<td>4.33</td>
<td>2.72</td>
<td>4.17</td>
</tr>
<tr>
<td>CMOD4=2 mm</td>
<td>2.11</td>
<td>4.05</td>
<td>2.48</td>
<td>3.77</td>
</tr>
</tbody>
</table>

Percentage Increase CMOD1 |
0% 48% -11% 41%

Percentage Increase CMOD2 |
0% 86% 13% 78%

Percentage Increase CMOD3 |
0% 89% 19% 82%

Percentage Increase CMOD4 |
0% 92% 18% 79%

### 7.4.2.4 Splitting Tension versus Flexure

Table 15 displays the ratio between the splitting tensile and flexural strength. Except for S1P1, the ratios of the different mixtures appear to be in the same order. This may suggest a relation between the splitting tensile and flexural strength that is not governed by the fibre dosage. Additionally, it shows a level of reliability of the results. Interestingly, the splitting tensile and flexural strength are close with regards to the absolute value. Furthermore, the lower ratio of mixture S1P1 could imply that the specimen performed worse in splitting tension or much better in flexural tension with respect to the other mixtures, assuming a fixed relation. It appears as if the latter assumption is more likely to be the case.
### 7.4.2.5 Flexure Preliminary Tests versus Flexure Secondary Tests

Table 16 displays the ratio between the flexural strength of the mixtures of the preliminary tests and the flexural strength of the mixtures of the secondary tests. The ratio corresponds with a size factor, which expresses the effect of the dimensions of the specimens on the strength properties. Except for S1, the ratio appears to be similar. Again, this may suggest a fixed relation that is independent of the fibre dosage and only dependent on the dimensions of the specimens that are being compared. It also implies a level of reliability of the results. Furthermore, the lower ratio of S1 may indicate that a subpar or superior flexural strength was recorded for mixture S1 in the secondary or preliminary tests, respectively. Based on the total results, it may be safe to assume that the lower ratio may be attributed a superior flexural strength for S1 in the preliminary tests.

### 7.4.2.6 Compression Strength versus Splitting Tensile Strength/Flexural Strength

It can be advantageous if there is a constant ratio between the compression strength and the tensile strength. The tensile strength can then be directly calculated from the compression strength. From Table 17 it can be observed that the ratios S/C for the different mixtures are in the same order, except for mixture S2P1. The same applies for the ratios F/C, except for S1.

### 7.4.2.7 Overview of Tested Specimens

**Specimen S1-02**

From the figures the shape of the crack, the fibre distribution and the orientation can be observed. The fibre orientation appears to be random. Many of the fibres are slanted and are positioned very low to
the cracked surface. Furthermore, areas with less fibres were observed. All of these may have affected the LOP or residual flexural strength negatively.

Figure 72: Specimen S1-02

Figure 73: Specimen S1-02
**Specimen S1-03**

It can be observed from the figures that the fibres appear to protrude sufficiently from the cracked surface. Furthermore, there is a homogenous distribution amongst the fibres. This may have resulted in the third prism recording a higher flexural strength than the second.

![Figure 74: Specimen S1-03](image)

**Specimen S2-02**

This specimen contains twice as much steel fibres as the previous specimens. Regarding the fibre orientation, the fibres appear to protrude sufficiently from the cracked surface. There are however areas where the fibres were positioned at a small angle relative to the cracked surface. Furthermore, the fibre distribution was favourable.
Specimen S1P1-01
From the images it can be observed that the fibres are protruding sufficiently from the cracked surface. However, the PVA fibres were difficult to observe. The fibre distribution appears to be relatively even and thus favourable.
Specimen S1P1-02
In this specimen the fibre distribution appears to be relatively homogenous. Furthermore, the fibres appear to be randomly oriented in many directions.

Figure 76: Specimen S1P1-01

Pouring direction

Figure 77: S1P1-02
Specimen S1P1-03

In this specimen, the fibres protruded sufficiently from the cracked surface. The dispersion of the fibres also appeared to be relatively even. Near identical LOP values were recorded for this particular specimen and its predecessor.
Specimen S2P1-01
The fibre distribution appears to be even. However, it can be clearly observed that there is a fair amount of fibres that are positioned unfavourably (the locations are shown within the red circles). This may have affected the residual flexural strength. It can be observed in the graph that lower residual strengths were recorded for this specimen. See Appendix D – Results of Secondary Tests.

Figure 80: Specimen S2P1-01

Figure 81: S2P1-01
Specimen S2P1-02

In this specimen the fibre distribution appears to be favourable. However, the fibre orientation appears to be subpar in some areas (the locations are shown within the red circles). A higher LOP was recorded for this specimen compared to the first and second. However, a lower residual strength was recorded compared to the third specimen.

Figure 82: Specimen S2P1-02

Figure 83: Specimen S2P1-02
Specimen S2P1-03
In this specimen the fibre distribution appears to be rather excellent. However, the orientation of the fibres in some areas were unfavourable. A higher residual strength was recorded for this specimen compared to the previous two.
7.4.3 Concluding Remarks

Compression Strength
It appears as if both PVA and steel fibres may have had a minimal effect in increasing the compression strength. Regarding the size effect, a fixed relation was obtained that is independent of the fibre dosage.

Splitting Tensile Strength
The splitting tensile strength is significantly influenced by the presence of the fibres. Moreover, the steel fibres appear to play a larger role in increasing the strength as opposed to the PVA fibres. However, in the case of S2P1, the PVA fibres appear to have contributed in increasing the strength more as opposed to the PVA fibres in mixture S1P1. This may imply an increase in collaboration with increasing steel fibre dosage.

Flexural Strength
The limit of proportionality (LOP) is positively influenced by the presence of fibres. Increasing the fibres may lead to an increase in LOP. At a lower dosage of steel fibres, PVA fibres appear to have a larger influence than at a higher steel fibre dosage. The same can be said of the residual flexural strengths. The flexural strength and splitting tensile strength appear to share a fixed relation, regardless of the fibre dosage. There also appears to be a fixed size factor between the flexural strength of the prisms used in the preliminary tests and the larger prisms used in the secondary tests.

As with the other properties, there also appears to be a fixed relation between the compression strength, the flexural strength and the splitting tensile strength.

7.5 Experimental Investigation - Phase III

7.5.1 Testing Methodology
The exact same methods that were utilize to test the material strengths in the first phase were also applied in this phase of testing. Contrary to the first phase, the amount of specimens per mixture was increased to twelve. In addition, the production process was altered in this phase. A mortar pump was used to fill the steel moulds with the fibre reinforced mortar as opposed to the use of a trowel in the first phase. Therefore, the fibre reinforced mortar was not poured in three layers into the steel moulds. In addition, a hand mixer was used to mix the fresh mortar as opposed to the mechanical mixer shown in Figure 39. Lastly, an extra mixture S1.5 was added for testing. This mixture contained 45 kg/m$^3$ of fibres. See Appendix G – Photos for photos.

The following properties were investigated:

3. Compression Strength in accordance with CUR-recommendation 24
4. Flexural Strength in accordance with CUR-recommendation 24 (ISO 679)
Twelve specimens per mixture were tested. First, the flexural tests were conducted. Subsequently, the tested specimens were used to determine the compression strength, all in accordance with CUR-Recommendation 24.

### 7.5.1.1 Compression Strength
The mortar compression strength $f_{mk}'$ was determined in accordance with CUR-Recommendation (ISO 679 and NEN 5968) at 7 days after casting. In this phase, 24 tests (2 x 12) per mixture were conducted. The maximum load was recorded.

### 7.5.1.2 Flexural Strength
The flexural strength was also determined using a three point bending test in accordance with CUR-Recommendation 24 (ISO 679). Twelve tests per mixture were conducted. The maximum load was recorded.

### 7.5.2 Results and Discussion
It was shown in the previous chapters that the fibres may lead to an improvement of the material properties. However, in this chapter it will be shown how the production process can significantly affect the flexural strength of fibre reinforced mortar. In this phase of testing it was observed that pumping of the mortar negatively influenced the flexural strengths.

#### 7.5.2.1 Compression Strength
Figure 85 and Table 18 display the average compression strength at 7 days of the pumped and regular mixtures. It can be observed that the compression strength increased noticeably with the addition of fibres. Again, the PVA fibres appear to have had no effect on the compression strength. Interestingly, it can also be observed that the pump appeared to have had a positive effect on the compression strength, though marginal. All pumped mixtures experienced an increase in strength compared to the regular mixtures. The maximum increase in strength was approximately 11.5% for S1. A possible reason for the increase in strength as a result of pumping could not be established. See Appendix E – Results of Tertiary Tests for a total overview of the results.
Table 18: Average compression strength at 7 days for the pumped and regular mixtures

<table>
<thead>
<tr>
<th>Compression [MPa]</th>
<th>Pumped Average</th>
<th>Pumped Increase wrt C</th>
<th>Regular Average</th>
<th>Regular Increase wrt C</th>
<th>Increase by pumping</th>
</tr>
</thead>
<tbody>
<tr>
<td>7 days</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>68.29</td>
<td>-</td>
<td>66</td>
<td>-</td>
<td>3.5%</td>
</tr>
<tr>
<td>S1</td>
<td>73.18</td>
<td>7%</td>
<td>65.6</td>
<td>-1%</td>
<td>11.5%</td>
</tr>
<tr>
<td>S1.5</td>
<td>76.30</td>
<td>12%</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>S2</td>
<td>75.99</td>
<td>11%</td>
<td>72.33</td>
<td>10%</td>
<td>5.1%</td>
</tr>
<tr>
<td>S1P1</td>
<td>-</td>
<td>-</td>
<td>70.18</td>
<td>6%</td>
<td></td>
</tr>
<tr>
<td>S2P1</td>
<td>78.63</td>
<td>15%</td>
<td>72.48</td>
<td>10%</td>
<td>8.5%</td>
</tr>
</tbody>
</table>

7.5.2.2 Flexural Strength

Figure 86 and Table 19 display the average flexural strength at 7 days of the pumped and regular mixtures. It can be clearly observed that the flexural strength of the pumped mixtures decreased significantly with increasing steel fibre dosage. A somewhat linear decreasing trend could be observed from mixture S1 to mixture S2. The recorded flexural strength of mixtures S2 and S2P1 was more or less the same. In addition, the pump appeared to have had a significantly negative effect on the flexural strength, particularly for the mixtures containing 60 kg/m³ of fibres. Moreover, mixtures S2 and S2P1 experienced the same decrease in strength as a result of pumping. Again, it can be observed that the PVA fibres, at the considered fibre dosage, had no effect on the flexural strength. Interestingly, the control specimen experienced a significant increase in strength compared to the control specimens of the regular mixtures. This could be attributed to the change in the production process.
7.5.2.3 Flexure versus Compression

Ratios between the flexural and compression strength for the different mixtures were computed to establish a relation, see Table 20. The ratio appeared to decrease with increasing steel fibre dosage. Figure 87 displays a trend of the ratios. It can be observed that the ratios followed a somewhat linear decreasing trend, which corresponds to the decrease in flexural strength of the mixtures. Furthermore, the ratio of the control specimen increased compared to the ratio of the control specimen of the regular mixtures, i.e. from 0.15 to 0.19. However, the ratios of the fibre reinforced mixtures decreased compared to the ratios of the regular mixtures. This shows that the ratio between the flexural strength and compression strength of the unreinforced mortar can be affected by the presence of fibres and the production process.

Table 20: Ratio flexure to compression for the pumped mixtures

<table>
<thead>
<tr>
<th>Steel fibre dosage [kg/m³]</th>
<th>C</th>
<th>S1</th>
<th>S1.5</th>
<th>S2</th>
<th>S2P1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexure [MPa]</td>
<td>13.07</td>
<td>12.18</td>
<td>11.20</td>
<td>9.62</td>
<td>10.02</td>
</tr>
<tr>
<td>Compression [MPa]</td>
<td>68.29</td>
<td>73.18</td>
<td>76.30</td>
<td>75.99</td>
<td>78.63</td>
</tr>
<tr>
<td>Ratio F/C</td>
<td>0.19</td>
<td>0.17</td>
<td>0.15</td>
<td>0.13</td>
<td>0.13</td>
</tr>
</tbody>
</table>
7.5.2.4 Overview of Tested Specimens
As in the previous phases, some of the tested specimens were observed and important facts were documented. This was done to help further clarify the results that were discussed in the previous chapter.

Specimen S1
Figure 88 below displays the cracked surface of the specimens. In the majority of the specimens it was discovered that the fibre distribution and orientation were subpar. It was therefore evident that production process had clearly affected the distribution and orientation of the fibres negatively. Furthermore, less fibres were present in the centre area of the specimens. This also occurred in the specimens of the other mixtures.
Specimen S1.5
The cracked surface of specimens are displayed in Figure 90. Again, the fibre distribution and orientation were subpar.

Specimen S2
Figure 157 displays the cracked surface of the specimens. Same discoveries were made as with the previous mixtures.
Specimen S2P1
The cracked surface of the specimens are displayed in Figure 91. It was discovered that the fibre distribution and orientation were the worse for this mixture. This could be a possible explanation as to why the lowest flexural strength was recorded for this mixture.
7.5.3 Concluding Remarks

Compression Strength
The results showed an improvement in compression strength as a result of pumping, though marginal. In addition, it was observed that a higher steel fibre dosage resulted in higher compression strength. The limit was 15% for the dosages under consideration, i.e. mixture S2P1.

Flexural Strength
The results provided compelling evidence that the flexural tensile strength was negatively affected by pumping the fibre reinforced mortar with the mortar pump under consideration in this research. Moreover, increasing the steel fibre dosage appeared to decrease the flexural strength significantly. The PVA fibres appeared to have had no affect on the flexural strength at the fibre dosage under consideration. Furthermore, the ratio between the flexural strength and compression strength showed a linear decreasing trend. This corresponds with the linear decreasing flexural strength of the mixtures with respect to the increasing fibre dosage.

Fibre Distribution and Orientation
The specimens were examined after testing. It was clearly observed that the fibre distribution and orientation were negatively affected as a result of the altered production process, i.e. hand mixing and pumping. This clearly resulted in the observed subpar strengths of the different mixtures. A possible reason for the subpar fibre distribution and orientation as a result of pumping could not established.
PART C

MODELLING THE BEHAVIOUR OF STEEL FBIRE RREINFORCED MORTAR
8 Introduction

The tensile properties of fibre reinforced cementitious composites (FRCC) is important in that it defines the tensile (post-cracking) behaviour. The tensile properties of FRCC can be expressed in a uniaxial tensile strength, splitting tensile strength and flexural tensile strength. These strengths and the corresponding tensile post-cracking behaviour can be studied experimentally or by means of numerical analysis (e.g.: FEM or excel). In this master project, the uniaxial tensile strength and corresponding tensile post-cracking behaviour was not investigated experimentally. Instead, it was chosen to study the tensile strength and post-cracking behaviour by means of three point bending tests on notched beams. Flexural tests has been employed by Kooiman [35] and Grünewald [36] in order to investigate the tensile behaviour of fibre reinforced concrete (FRC) and fibre reinforced self compacting concrete (FRSCC), respectively. In their studies, the uniaxial strength was determined using the results of the flexural tests. However, in order to do so, an inverse modelling procedure has to be applied. This can be based on a modified version of the multi-layer design model originally developed by Hordijk [37], in which the results of the experimental tests are used as input.

As previously said, this model is based on an inverse modelling procedure where backward calculations are conducted in order to simulate load-crack opening displacement diagrams. These computed curves are then compared to the load-crack opening displacement diagrams that were acquired from the flexural tests. Subsequently, a parametric study was conducted by varying certain input parameters (including an assumed uniaxial strength) in order to study the effect of the parameters on the load-crack opening displacement diagrams and to determine optimal values concerning these parameters.

This part of the report is structured as follows. First, modelling of the tensile behaviour of SFRM is addressed in Chapter 9. In this chapter, a constitutive model for cementitious material, such as stress-strain/crack width model, is addressed. This model have been used frequently to describe the uniaxial and post-cracking behaviour of plain and steel fibre reinforced cementitious composites. In addition, the theoretical background behind the inverse modelling procedure is discussed. Subsequently, the basic principles of the inverse modelling procedure is expanded on. The material properties of SFRM are addressed in Chapter 10. The material and geometrical properties of the SFRM mixtures and specimens, respectively, are used as input data for the inverse modelling. First, the properties from experimental results are addressed, followed by other properties that were obtained via references. Chapter 11 continues with analyses of the material behaviour. The unknown properties that define the stress-strain/crack width relations are investigated and optimized. This was done through a parametric study of the parameters that influence the tensile post-cracking behaviour. Finite element analyses of the material behaviour was also carried out. The properties obtained from the results of the inverse modelling procedure were used as input for the finite element model. These properties were optimized by means of finite element analyses.
9 Modelling the Tensile Behaviour of SFRM

9.1 Constitutive Model for Cementitious Material

Different constitutive models exist that can be used to describe the softening behaviour of cementitious materials. Models such as the 'stress-crack' width method, 'crack band theory' and the 'stress-strain' method exist [38-40]. The choice of the model may depend on availability and amount of input parameters. Also, the applicability of the model for structural design purposes may play a part. Kooiman studied these different models and showed that the stress-crack width method meets the criteria best for a constitutive model and is most suitable for structural design purposes amongst the three. The criteria that Kooiman used are listed as the following:

**Criteria for a suitable design model for structural design purposes according to Kooiman [35]:**

- The mean softening behaviour from test results should be predicted and fitted with sufficient accuracy by the material model.
- Size-effects should be included in the model if necessary.
- Reduction factors for sustained loading conditions should be available for use if necessary.
- Reduction factors for varying fibre efficiency related to the structural application and its production process should be available for use if necessary.
- Scatter should be taken into account and processed in the model: characteristic values should be determined from mean values and the quantified scatter.
- Probabilistically determined safety factors should be used to transform the representative values into design values.

Additionally, an important advantage of the stress-crack width method as opposed to the other methods is its usefulness in making the behaviour of FRCC on a material level comprehensible. The method is based on the Fictitious Crack Model (FCM) of Hillerborg et al. [38]. According to FCM, the stress-displacement relationship consists of a stress-strain relation for the linear-elastic tensile behaviour prior to cracking and a stress-crack width relation for the softening behaviour after cracking. This is illustrated in Figure 92 below. In the fictitious crack model it is assumed that stresses are transferred over a certain length from the fictitious crack tip. This takes place in a 'cohesive zone' until a critical crack width $w_0$ is reached. The length of the zone can be increased by adding fibres to the mix. The length in this case is then dependent on the length of the crack that is bridged by the fibres, see Figure 93.
Figure 92: Stress-displacement relationship according to the Fictitious Crack Model of Hillerborg at al. [38]

Figure 93: Fictitious Crack Model [38]

There are two different approaches to FCM. The first is a micromechanical approach developed by Li et al. [41]. They developed a semi-analytical model to describe the post-cracking behaviour of FRCC. This method makes use of so-called micro-mechanical parameters, which can be determined experimentally. The second is the inverse analysis approach. The focus of this research is placed on the second approach, which is simple and straightforward. This is further addressed in the following subchapter.

9.2 Stress-Crack Width Relation

The stress-crack width relation can be schematized in different ways. Moreover, the relation is described by a number of degrees of freedom. Research by Kooiman and Grünewald [35, 36] have shown that stress-crack width relation with at least four degrees of freedom provides an adequate fitting curve for the tensile post-cracking behaviour. In their research, a bilinear stress-crack width relations were defined. This is illustrated in Figure 94.
The bi-linear relationship can be expressed in the following parameters and equations:

\[
\sigma_{ct} (w) = f_{ct,ax} - \frac{w}{w_c} f_{ct,ax} - f_{ct,eq,bi} \text{ for } w \leq w_c \tag{9.1}
\]

\[
\sigma_{ct} (w) = f_{ct,eq,bi} \left( \frac{w_0 - w}{w_0 - w_c} \right) \text{ for } w_c < w \leq w_0 \tag{9.2}
\]

Where

- \( f_{ct,ax} \) = uniaxial tensile strength [N/mm\(^2\)]
- \( f_{ct,eq,bi} \) = bilinear equivalent post-cracking strength [N/mm\(^2\)]
- \( w_0 \) = critical crack width [mm]
- \( w_c \) = characteristic crack width [mm]

The complete post-cracking behaviour from crack initiation up to the critical crack width \( w_0 \) can be described by the initially assumed and final determined stress-crack width relation. This approach is more practical, because less parameters are required and are to be verified experimentally. On the other hand, the determination of a stress-crack width relation via this method does not provide a full understanding of the material's (post-cracking) behaviour. Only through trial and error can this be achieved with this method.

Kooiman further developed a modified version of the stress-crack width relation in order to create a compatible stress-strain/crack width opening displacement diagram that can describe the flexural tensile behaviour prior to and after cracking. This is discussed in Chapter 11.

### 9.3 Inverse Modelling

The uniaxial or flexural tensile behaviour of SFRM can be modelled using different methods. One method is the inverse modelling procedure. The following description of Van Mier [42] explains clearly the principle of the inverse modelling procedure. It is described as follows:

In an inverse modelling procedure, a softening diagram is assumed, and in a numerical analysis of the fracture geometry under consideration, the deviation between the computational and experimental results is calculated. Depending on this deviation and the permissible error, the analysis is repeated...
using a modified softening diagram. The procedure is repeated until the error has become small enough to fall within predetermined boundaries.

This procedure can be used to obtain a suitable stress-crack width opening relation for the SFRM material. Hordijk [37] developed the multi-layer simulation method that is based on this inverse modelling approach. This is discussed in Subchapter 9.4.

The inverse modelling procedure is based on four steps that are carried out in a particular order as shown in the flow chart, see Figure 95. These four steps or levels are explained below.

![Flow Chart](image)

**Figure 95: Inverse Modelling Procedure [Roelfstra & Wittman, 1986]**

**Level I: input Level**
On this level the geometrical properties of the specimens and assumed uniaxial material properties are set. The geometrical properties, which concerns the geometry of the specimens, consist of the effective beam depth $h_{ig}$ (depth of beam above notch), the beam width $b$ and the span length $L_d$. Furthermore, the uniaxial material properties consist of the uniaxial compressive and uniaxial tensile strength. The mean uniaxial compressive strength is acquired from the compressive tests on cubes, whereas uniaxial tensile strength is assumed as a factor multiplied by the mean splitting tensile strength. The mean splitting tensile strength was also determined from tests on cubes.

The input level of the inverse modelling procedure is therefore defined by three material relations:

- A stress-strain diagram for SFRM in compression
- A stress-strain diagram for SFRM in linear elastic tension
- A stress-crack width relation for the post-cracking behaviour of SFRM.
Level II: simulation level
This is the level where the cracked cross-section of the notched beam is numerically analysed by means of the inverse modelling approach. The inverse approach is carried through the multi-layer model of Hordijk [37]. This model was programmed in excel using excel VBA. The multi-layer model is discussed in Subchapter 9.4.

Level III: accuracy check
On the third level an accuracy check is conducted of the computed load-crack opening displacement diagram. The deviation of the toughness D (area under the load-displacement diagram) and the load P (up to a displacement $\delta_m$) between the computed and measured diagrams is calculated. The analysis is repeated if the deviation of the toughness and loads exceeds a permissible error of 10%. The parameters of the stress-crack width diagram are modified until the error is smaller than the permissible error.

For the load $P$, the maximum allowed deviation between the measured load $P_m(\delta)$ and computed load $P_c(\delta)$ up to a displacement $\delta^*_m$ is set to 10%. The load $P_m(\delta^*_m)$ that corresponds with this displacement equates to a load that is 75% of the maximum measured load $P_{m,\text{max}}$. See Figure 96. The deviation can be calculated as follows:

$$\text{If } P_m(\delta) > P_c(\delta) \text{ then } \Delta P = P_m(\delta) - P_c(\delta) \rightarrow \frac{\Delta P}{P_m(\delta)} < 0.1$$ (9.3)

$$\text{If } P_m(\delta) < P_c(\delta) \text{ then } \Delta P = P_c(\delta) - P_m(\delta) \rightarrow \frac{\Delta P}{P_c(\delta)} < 0.1$$ (9.4)

Level IV: output level
This is the output level where the parameters of the stress-crack width relation diagram meet the required accuracy.

9.4 Multi-layer Modelling Procedure
Hordijk [37] developed the multi-layer material model, which is based on the inverse modelling approach. He used it to study the flexural behaviour of plain concrete. Kooiman [35] employed a modified version of this model to study the flexural behaviour of fibre reinforced concrete. This modified version was used in this thesis. The difference between the modified version and Hordijk's version is found in the type of displacement considered for the model. Hordijk considered the vertical
displacement in his model whereas Kooiman considered the crack mouth opening displacement. One of the advantages of this model is its simplicity, which allows it to be easily programmed in a spreadsheet. Figure 97, Figure 98 and Figure 99 below displays the basic principles of the multi-layer model. According to the first principle displayed in Figure 97, a beam is split into two halves that are connected by springs that represent the behaviour of a finite layer. In addition, a linear displacement distribution is assumed over the effective beam depth. According to the second principle displayed in Figure 98, a deformation $\delta$ is determined for each spring by computing the average deformation in the related layer. For each layer the stress can then be determined from the stress-crack width opening displacement diagram, see Figure 97. Subsequently, for each layer the normal force $N$ and the bending moment $M$ are computed from the linear deformation distribution. Adding the contributions of all the layers should lead to an equilibrium of the internal force $N$ and to a total internal bending moment. Equilibrium is achieved if $N$ is nihil. In addition, the external load can be determined from the internal bending moment, which is equal to the external bending moment caused by the external load, see Figure 98. In the third and final principle the crack opening displacement at the notch is incrementally increased with small steps. Iteration is then performed to accomplish equilibrium of internal forces. This is done by adjusting the displacement at the top of the cross-section until equilibrium of internal forces is achieved. From the equilibrium of the internal forces and bending moment, the external load can then be computed. This whole process is repeated until a total load-crack opening displacement diagram is simulated, see Figure 99.

![Diagram](image-url)
The displacement $\delta_{\text{notch}}$ represents the displacement at the top of the notch. However, the experimental load-displacement diagrams are based on the displacements (CMOD) at the bottom of the notch. Therefore, the computed displacements of the multi-layer model were recalculated to represent the displacements at the bottom of the notch using the following equation:

$$CMOD = \delta_{\text{notch}} + \frac{\delta_c - \delta_{\text{notch}}}{(h-a)} \times a$$

(9.5)

Where:
- CMOD: Crack Mouth Opening Displacement [mm]
- $\delta_{\text{notch}}$: Crack Tip Opening Displacement [mm]
- $\delta_c$: displacement at the top of the beam [mm]
- $h$: height of the beam (150 mm) [mm]
- $a$: height of the notch (25 mm) [mm]
10 Properties of Steel Fibre Reinforced Mortar

In this chapter the relevant properties that are set to the input level of the inverse modelling procedure are discussed. To utilize the multi-layer model input parameters that describe the stress-crack width relation are to be defined. These input parameters consist of geometrical and material properties of SFRM that may be determined from experiments, design guidelines or from references. However, before the input parameters are expanded on, the mixtures under consideration are first discussed. Three point bending tests were conducted on four mixtures. However, only two of the mixtures are further investigated. Thereafter, the determined material properties are expanded on. Not all material properties were determined from the experiments. Therefore, some references were consulted in order to determine the unknown properties. This is addressed in the third subchapter.

10.1 Mixtures

In this master project an experimental investigation of the material behaviour of steel fibre reinforced thixotropic mortar was conducted. Twelve different mixtures were initially investigated. The investigated properties included the compression and flexural strength. Four mixtures were further investigated in order to study the post-cracking flexural tensile behaviour. In addition, the splitting tensile strength was determined. These four mixtures are displayed below.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Mixture ID</th>
<th>Volume fraction Steel (%)</th>
<th>Volume fraction PVA (%)</th>
<th>Fibre dosage (kg/m^3)</th>
<th>Total fibre dosage (kg/m^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>S1</td>
<td>0.38</td>
<td>0.00</td>
<td>30</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>S2</td>
<td>0.76</td>
<td>0.00</td>
<td>60</td>
<td>0.00</td>
</tr>
<tr>
<td>3</td>
<td>S1P1</td>
<td>0.38</td>
<td>0.30</td>
<td>30</td>
<td>3.90</td>
</tr>
<tr>
<td>4</td>
<td>S2P1</td>
<td>0.76</td>
<td>0.30</td>
<td>60</td>
<td>3.90</td>
</tr>
</tbody>
</table>

The results of the experiments showed that both S1 and S1P1 resulted practically in the same residual flexural capacity. The same was discovered for S2 and S2P1. It was therefore concluded that the PVA fibres provided little to no contribution to the flexural strength for the considered PVA fibre dosage. For this reason it will suffice to study the tensile post-cracking behaviour of just two mixtures and, in addition, the influence of the steel fibre dosage can be investigated. The results of S1P1 and S2P1 were used as input for the inverse modelling procedure, since complete valid tests results were obtainable only for these two mixtures.

10.2 Material Properties

As mentioned in the previous subchapter, mixtures S1P1 and S2P1 are considered for further analysis. First, the properties that were obtained from the experiments are addressed followed by the properties determined via references.
10.2.1 Properties Obtained From Experimental Results

The average of the results of the experimental tests for these two mixtures are summarised in Table 22. The table displays the average of the compression strength, splitting tensile strength and of the maximum load from the flexural test. The related Crack Mouth Opening Displacements (CMOD) at the maximum loads are also displayed. To analyse the post-cracking behaviour of SFRM, it is common practice to determine toughness values $D$ and the corresponding energy absorption capacity $G_{fc}$. Moreover, the toughness values and energy absorption capacity express the ductility of SFRM. Therefore, the energy absorption capacity $G_{fc}$ for both mixtures is listed. $G_{fc}$ is calculated as the toughness $D$ (area under the load-CMOD diagram) up to a displacement of 2 mm divided by the cross sectional area of the specimen above the notch ($A_{eff}$).

$$G_{fc} = \frac{D}{A_{eff}} = \frac{D}{b \cdot h_{lig} \left[ \frac{N}{mm} \right]}$$  \hspace{1cm} (10.1)

Where

$D$  toughness value [Nmm]

$b$ width of specimen (150 mm) [mm]

$h_{lig}$ effective beam depth (125 mm) [mm]

In addition to the averages, a coefficient of variation (CoV) or variation $V$ was computed for the maximum load and $G_{fc}$. The coefficient of variation $V_{g}$ for $G_{fc}$ expresses the scatter of post cracking energy. Table 22 shows that a large scatter was obtained for mixture S2P1. This is notably larger than the measured scatter for S1P1. The scatter is also necessary to determine characteristic and subsequently design values for SFRM.

<table>
<thead>
<tr>
<th>Mix</th>
<th>ID</th>
<th>$f_{ccm}$ [N/mm$^2$]</th>
<th>$f_{ctm,spl}$ [N/mm$^2$]</th>
<th>$P_{max}$ [kN]</th>
<th>$V_P$ [%]</th>
<th>$\delta_{pmax}$ [mm]</th>
<th>$G_{fc}$ [N/mm]</th>
<th>$V_G$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>S1P1</td>
<td>61.91</td>
<td>5.01</td>
<td>18.8</td>
<td>0.38%</td>
<td>0.0238</td>
<td>1.03</td>
<td>2.77%</td>
</tr>
<tr>
<td>4</td>
<td>S2P1</td>
<td>62.77</td>
<td>5.85</td>
<td>19.0</td>
<td>3.89%</td>
<td>0.0243</td>
<td>1.44</td>
<td>17.28%</td>
</tr>
</tbody>
</table>

Table 23: Properties of fibres

<table>
<thead>
<tr>
<th>Fibre Type</th>
<th>Length $L_f$ [mm]</th>
<th>Diameter $\phi_f$ [mm]</th>
<th>Aspect Ratio $L_f/\phi_f$ [-]</th>
<th>Density $\rho$ [kg/m$^3$]</th>
<th>Tensile Strength $f_t$ [MPa]</th>
<th>Elastic Modulus $E$ [GPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>13</td>
<td>0.20</td>
<td>65</td>
<td>7850</td>
<td>2600</td>
<td>210</td>
</tr>
<tr>
<td>PVA RF 1000</td>
<td>15</td>
<td>0.31</td>
<td>48</td>
<td>1300</td>
<td>1000</td>
<td>29</td>
</tr>
</tbody>
</table>

Table 24: Geometrical properties of notched beam

<table>
<thead>
<tr>
<th>Length Beam $L$ [mm]</th>
<th>Span Length $L_d$ [mm]</th>
<th>Beam Width $b$ [mm]</th>
<th>Notch height $a$ [mm]</th>
<th>Beam Depth $h$ [mm]</th>
<th>Effective beam depth $h_{lig}$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>600</td>
<td>500</td>
<td>150</td>
<td>25</td>
<td>150</td>
<td>125</td>
</tr>
</tbody>
</table>
Figure 100 displays the average results of the measured load-CMOD diagrams for mixtures S1P1 and S2P1. It is worth mentioning that prior to cracking (linear elastic stage) both mixtures achieved approximately the same maximum load at practically the same displacement. The reason is that the fibres are predominantly activated after cracking has occurred. Therefore, prior to cracking the stresses are transferred almost entirely by the matrix. Also, the compression strength for both mixtures are practically the same. It is also worth mentioning that the tail of the load-displacement diagrams is roughly horizontal up to a displacement of 2 mm for both mixtures. In addition, the horizontal tail occurs within a 'small displacement range' following the maximum load. This 'small range' and the horizontal tail both have had a pronounced effect on the input parameters of the stress-crack width relation that is determined with the inverse modelling procedure. This will be addressed in Chapter 11.

![Load-Displacement diagram mixture S1P1/S2P1](image)

It was mentioned previously that the input parameters for the inverse modelling procedure consists of parameters that are defined by setting three material relations; the stress-strain relation in compression, the stress-strain relation in linear elastic tension, and a stress-crack width relation for the post-cracking behaviour.

10.2.2 Properties Determined From References

10.2.2.1 Compression Elastic Modulus; influence of fibre aspect ratio and dosage on E-modulus

Although the compression strength was determined from cube tests, the linear elastic strain limit $\varepsilon_{c3}$ and ultimate strain limit $\varepsilon_{c3u}$ were unknown. In order to determine these values, the Young’s modulus of the material was needed. The Young’s modulus was acquired from Young’s modulus test conducted by Van Keulen [2]. This was done by compressing cylinders up to a compression stress of 15 N/mm$^2$, which was well within the 1/3 of the maximum compressive stress that is commonly used to determine the elastic modulus. The cylinder specimens consisted of a similar SFRM mixture and were tested at an age of 32 days and older. The matrix was the same as the matrix used in this research, i.e. Thixomortar K70. The fibre dosage was also the same. However, the applied steel fibres were slightly thinner than the
steel fibres applied in this research; i.e. 0.16 mm in diameter with an aspect ratio of 81 as opposed to 0.2 mm in diameter with an aspect ratio of 65.

Misba et al. [43] and Ezeldin et al. [44] included a studied the modulus of elasticity of steel fibre reinforced concrete. It was shown that the elastic modulus of SFRC may be affected by the fibre aspect ratio. Moreover, it was shown that a higher fibre aspect ratio may lead to a higher elastic modulus. Fanella et al. [45] studied the stress-strain properties of mortar in compression reinforced with amongst others straight smooth steel fibres. The properties and characteristics of the mortar are listed in Figure 101. Mix 2 resembles the thixotropic mortar K70 applied in this research. It was concluded that an increase in fibre aspect ratio at a constant fibre dosage leads to an improvement in the post-peak behaviour in compression. In addition, the aspect ratio and fibre length may increase the strain at the peak load, see Figure 102.

Given the fact that in Van Keulen's research the secant modulus was determined at a stress equal to 1/3 of the maximum compression stress, it is safe to assume, according to the findings of Fanella et al. [45], that the experimental results of the secant modulus corresponds with a tangent modulus. Therefore, it is also safe to assume that using the data from Van Keulens research may lead to an overestimation of the secant elastic modulus at peak stress $E_{sp}$ (see Figure 103). The tangent modulus appeared to remain unaffected by the fibre aspect ratio. The estimated values for the tangent modulus and secant modulus at peak stress based on the diagrams in Figure 102 are listed in Table 25. From these values a ratio between the tangent modulus and secant modulus at peak stress was calculated. For the control specimen the ratio was 1.08. The ratio increases with increasing fibre aspect ratio, see Figure 104. Eurocode 2 [46] proposes a ratio of 1.05 for concrete. In the eurocode, however, the secant modulus is approximated for a value in the range of $\sigma_c=0$ and $0.4f_{c,m}$ as opposed to the peak stress.
Table 1: Characteristics of mortar mixtures; strength determined at 7 days [45]

<table>
<thead>
<tr>
<th>Mix number</th>
<th>Sand-cement ratio</th>
<th>Water-cement ratio</th>
<th>Mix design, cement:sand:water</th>
<th>Strength, ksi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3</td>
<td>0.5</td>
<td>1:3:0.5</td>
<td>7.2 (49.7)</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>0.5</td>
<td>1:2:0.5</td>
<td>8.5 (58.6)</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>0.35</td>
<td>1:1:0.35</td>
<td>9.9 (68.3)</td>
</tr>
</tbody>
</table>

Figure 101: Influence of the aspect ratio of fibres on the stress-strain curve for mix 2 [45]

Figure 102: Secant and Tangent Modulus of Elasticity; $E_t$ and $E_{sp}$ are equal up to the proportional limit of the material [47]
Furthermore, Fanella [37] showed that increasing the fibre dosage may improve the post-cracking behaviour. However, the strain at the peak load appears to remain unaffected by the change in fibre dosage, see Figure 105. This is also evident in the load-crack opening displacement diagram of the mixtures of this research, see Figure 100. Therefore, these findings may imply that the compressive secant modulus is influenced by the fibre aspect ratio and not the fibre dosage. Furthermore, the tangent modulus remained unaffected.

**Table 25: Secant Modulus at peak stress versus Tangent Modulus; values are estimated from the curves in Figure 102**

<table>
<thead>
<tr>
<th>Fibre Aspect Ratio</th>
<th>Peak Strain</th>
<th>Peak Stress [MPa]</th>
<th>Et [MPa]</th>
<th>Esp [MPa]</th>
<th>Ratio Et/Es</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.0034</td>
<td>58.7</td>
<td>18630</td>
<td>17250</td>
<td>1.08</td>
</tr>
<tr>
<td>47</td>
<td>0.0040</td>
<td>58.7</td>
<td>18630</td>
<td>14663</td>
<td>1.27</td>
</tr>
<tr>
<td>83</td>
<td>0.0044</td>
<td>58.7</td>
<td>18630</td>
<td>13330</td>
<td>1.40</td>
</tr>
<tr>
<td>100</td>
<td>0.0058</td>
<td>62.1</td>
<td>18630</td>
<td>10707</td>
<td>1.74</td>
</tr>
</tbody>
</table>

**Figure 104: Ratio Et/Es versus fibre aspect ratio**

**Figure 105: Influence of volume fraction on the stress-strain curve for mix 2 [45]**
10.2.2.2 Compression Elastic Modulus; time development of elastic modulus

Carmichael [48] studied the development of the Young's modulus in time and the relationship between the Young's modulus and compressive strength for early age concrete. It was shown that the Young’s modulus linearly increases when older than 10 days, see Figure 106. Yoshitake et al. [49] also studied the development of the Young's modulus of early age concrete and mortar in time. The considered mortar had similar mixture proportions as the mortar applied in this study. It was shown that both mortar and concrete experienced a rapid development in the first 4 days followed by a linear increase of the Young’s modulus, see Figure 107. Based on these findings it may be safe to assume a linear behaviour to extrapolate the results of Van Keulen in order to determine the elastic modulus of the considered mixtures. Since the three point bending tests were conducted at an age of 8 days after casting, a corresponding elastic modulus was calculated from the measured results of Van Keulen, see Figure 108.

\[ E = 3.3 \times 10^6 e^{0.018t} \]

Figure 106: Young's modulus versus age [35]

Figure 107: Tensile and compressive secant E-moduli (under 1/3 of maximum stress) versus age of concrete and mortar
10.2.2.3 Compression; elastic and ultimate strain limits

Since the inverse model approach requires the linear elastic strain instead of the strain at the peak load to be used as input, the linear elastic strain is calculated as follows:

\[ S1P1: \sigma = \varepsilon \cdot E \rightarrow \varepsilon_{\text{c3}} = \frac{f_{\text{ccm}}}{E_t} = \frac{61.91}{18245} = 3.39 \times 10^{-3} = 3.39\% \]  \hfill (10.2)

\[ S2P1: \sigma = \varepsilon \cdot E \rightarrow \varepsilon_{\text{c3}} = \frac{f_{\text{ccm}}}{E_t} = \frac{62.77}{18245} = 3.44 \times 10^{-3} = 3.44\% \]  \hfill (10.3)

Where

\[ \varepsilon_{\text{cp}} \] strain limit at peak stress
\[ f_{\text{ccm}} \] linear elastic stress limit [MPa]
\[ E_{\text{c}} \] compressive elastic modulus [MPa]; determined from Figure 108

For further calculations, a linear elastic strain limit of 3.4 % is assumed for both mixtures. The ultimate strain limit was fixed to 10 %. The ultimate strain limit did not have any influence on the load-cracking mouth opening displacement diagrams.

10.2.2.4 Tensile Elastic Modulus

According to CEB-FIP model code 1990 [50] the tension elastic modulus may be assumed as equal to the compressive tangent modulus for concrete. Yan [51] studied the uniaxial tensile behaviour of SFRM and the effect of fibre length and fibre volume fraction of straight steel micro fibres on the tensile strength and elastic modulus of the composite. It was shown that the tangent elastic modulus remains unaffected when increasing the fibre dosage or length. Gopalaratnam et al. [52] also investigated the tensile failure of SFRM, reinforced with straight steel fibres with an aspect ratio of 62.5. Both compressive and tensile tangent moduli were measured. The results showed that the compressive and tensile tangent modulus are similar. The compressive tangent modulus was in the same range as the measured elastic modulus of Van Keulen.

![Figure 108: compressive elastic modulus of mortar](image-url)
10.2.2.5 Tensile Strength

The uniaxial tensile strength is used as input for the tensile linear elastic behaviour in the inverse analysis procedure. Gopalaratnam et al. [52] showed that the fibre dosage affects the uniaxial tensile strength, though marginal. When the fibre dosage was doubled from 0.5% to 1% (by volume), the tensile strength increased by a mere 7%.

The uniaxial strength is related to the splitting tensile strength \( f_{\text{fctm, spl}} \). EC 2 [46] and the Model Code 1990 [35] propose an uniaxial to splitting tensile strength ratio of 0.9. Körmeling [53] also studied the behaviour of SFRC in tension. He showed that in the case of SFRC, the ratio is in the range of 0.7 to 0.8. It will be shown in the following chapter that these ratios could lead to an overestimation of the uniaxial strength of SFRM mixture S2P1.

Furthermore, the mixtures under consideration showed an increase in splitting tensile strength of roughly 17% after doubling the steel fibre dosage from 30 to 60 kg/m\(^3\) (0.38% to 0.78% by volume). However, the load-displacement diagram showed no increase in flexural peak load. In fact, the peak load remained almost unaffected, see Figure 100. For this reason it is assumed that the uniaxial tensile strength is more or less equal for both mixtures. This is investigated in the following chapter.

10.2.2.6 Tensile Elastic Strain Limit

Gopalaratnam [52] obtained an elastic strain of 0.188 \( \text{‰} \) for a mixture that is similar to the mixtures under consideration in this research. This value for the elastic strain was therefore used as a starting point for the elastic strain of mixtures S1P1 and S2P1. Assuming that the tensile elastic modulus is equal to the compressive elastic modulus as suggested in the Model Code, the uniaxial tensile strength was calculated as follows:

\[
\sigma = \varepsilon \cdot E \rightarrow \sigma = 0.188 \cdot 10^{-3} \times 18245 = 3.43 \text{ MPa}
\]  

(10.4)

As discussed in the previous subchapter, the uniaxial tensile strength is assumed to be the same for both mixtures. A ratio between the uniaxial and splitting tensile strength was calculated given the fact that the uniaxial tensile strength relates to the splitting tensile strength. The ratios for both mixtures were calculated as follows:

For S1P1 \( \rightarrow \alpha = \frac{f_{\text{fctm, ax}}}{f_{\text{fctm, spl}}} = \frac{3.43}{5.01} = 0.68 \)  

(10.5)

For S2P1 \( \rightarrow \alpha = \frac{f_{\text{fctm, ax}}}{f_{\text{fctm, spl}}} = \frac{3.43}{5.85} = 0.59 \)  

(10.6)

The results will show that these assumptions pertaining to the elastic strain, tensile elastic modulus and the corresponding uniaxial tensile strength resulted in a slight underestimation of the load-displacement diagrams of the mixtures. See Chapter 11.

10.3 Concluding Remarks

The material and geometrical properties of SFRM mixtures are used as an input for the inverse analysis. The properties may be determined experimentally or acquired from other sources. In the case of this research the compressive strength, splitting tensile strength, flexural tensile strength, flexural peak load, displacement at peak load and energy absorption capacity were acquired from the experimental results. However, the compressive elastic modulus, compressive elastic and ultimate strain limit, the tensile
elastic modulus and tensile elastic strain limit were determined from other sources. In the case of the
compressive elastic modulus, the measured results from Van Keulen's research were used.

It was shown by other researchers that the compressive elastic modulus at peak stress may be affected
by fibre's aspect ratio, but that the compressive elastic modulus within 1/3 of the maximum
compressive stress remains unaffected. Moreover, this elastic modulus corresponds more with a
tangent modulus of elasticity and may be used as input for the inverse modelling procedure.
It was also shown that the tensile tangent elastic modulus is equal to or in the range of the compressive
tangent modulus. Furthermore, the uniaxial tensile strength that is used to determine the linear elastic
behaviour is related to the splitting tensile strength.
11 Analysis of Flexural Tensile Behaviour of SFRM Cross-Section

This chapter addresses the other input parameters that define the uniaxial stress-strain and the bilinear tensile softening relation for the SFRM mixtures that were attained via the inverse analysis. These parameters were not obtained via experiments or from references as in the case of the parameters addressed in the previous chapters. Instead, these were determined through an iteration process with the inverse analysis procedure. Initial assumptions were made based on values proposed by Kooiman. Subsequently, a parametric study was conducted to investigate the effect of the parameters determined through inverse analysis procedure on the load-displacement diagrams and to obtain optimal values for the parameters. This chapter concludes with remarks concerning the findings.

11.1 Parametric Study

The influence of the elastic modulus on the flexural stiffness and the parameters of the tensile softening stress-crack width relation on the tensile post-cracking behaviour of SFRM is addressed in this subchapter. The parameters are as follows:

- Elastic Modulus $E_t$
- Uniaxial tensile strength $f_{ct,ax}$
- Equivalent bilinear post-cracking strength $f_{ct,eq,bil}$
- Characteristic crack width $w_c$
- Critical crack width $w_0$

11.1.1 Influence of Elastic Modulus on Stiffness

From the analyses it was discovered that the previously calculated tangent elastic modulus of $E_t=18245$ MPa may have been an underestimation of the true tangent elastic modulus. The initial value was determined from measured results through linear interpolation according to trends found in external references. See Subchapter 10.2.2. However, other sources show [54] that the elastic modulus of concrete or cementitious materials with rapidly developing strength also undergoes a rapid (non-linear) development in the early age, see Figure 110 and Figure 111. At 8 days the elastic modulus is almost equal to the value at 28 days. Therefore, based on this it was assumed that the initial (tangent) elastic modulus of the fibre reinforced mortar at 8 days is more or less equal to the estimated elastic modulus at 28 days, i.e. $E_{t,28}=24870$ MPa. The results shows that $E_{t,28}$ indeed led to a better initial stiffness of the load-displacement diagram compared to $E_{t,8}$, see Figure 112. However, it can also be observed that the
estimated E-modulus $E_t,28$ resulted in a slightly higher initial stiffness of the load-displacement diagram. An E-modulus value of approximately $E_t,8 = 23000$ MPa resulted in the best fit.

Figure 110: Development of E-modulus, tensile and compression strength over time of rapidly developing concrete [54]

Figure 111: Non-linear development of E-modulus [54]

Figure 112: Load-displacement diagram up a displacement of CMOD = 0.2 mm
Since a different elastic modulus was applied, the compressive elastic strain was recalculated based on the optimized elastic modulus as follows:

\[ S1P1: \sigma = \varepsilon \cdot E \rightarrow \varepsilon_{c3} = \frac{f_{cm}}{E_t} = \frac{61.91}{23000} = 2.69 \cdot 10^{-3} = 2.69\% \ (11.1) \]

\[ S2P1: \sigma = \varepsilon \cdot E \rightarrow \varepsilon_{c3} = \frac{f_{cm}}{E_t} = \frac{62.77}{23000} = 2.73 \cdot 10^{-3} = 2.73\% \ (11.2) \]

### 11.1.2 Influence of the Uniaxial Tensile Strength

The effect of the uniaxial tensile strength on the load-displacement diagrams of mixtures S1P1 and S2P1 is displayed in Figure 114 and Figure 115, respectively. In the case of S1P1, it can be observed that the peak load was significantly affected by the uniaxial tensile strength, whereas the tail of the curve remained relatively unaffected for the considered ratios between the uniaxial and splitting tensile strength. For S1P1, a ratio of approximately 0.71 resulted in the best fit for the load-displacement diagram. The curve met the requirements of the accuracy check of a maximum deviation in load P and toughness D of 10% between the measured and computed load-displacement diagram.
Figure 114: Influence of the uniaxial tensile strength on the load-displacement diagram of mixture S1P1

Figure 115 shows that the uniaxial tensile strength affected both the peak load and the tail of the curve for the considered parameters of mixture S2P1. Furthermore, a ratio of 0.62 between the uniaxial and splitting tensile strength resulted in a good fit.

11.1.3 Influence of the Critical Crack Width

Kooiman proposed a ratio between the critical crack width \( w_0 \) and the fibre length \( L_f \) in the range of 0.33 to 0.425. It was observed that the critical crack width affected the peak load slightly within the given range of ratios. The tail of the curve was also slightly affected. A ratio of 0.35 resulted in a good fit.
Mixture S2P1 showed a similar behaviour concerning the critical crack width $w_0$. A ratio of 0.33 resulted in a good fit.

11.1.4 Influence of the Characteristic Crack Width

Figure 118 displays the effect of the characteristic crack width $w_c$ on the load-displacement diagram. It was observed that the characteristic crack width $w_c$ had a pronounced effect on the peak load and the convex shape of the load-displacement diagram. The tail of the curve remained unaffected. A ratio $w_c/w_0$ in the range of 1/42 resulted in a good fit.
The characteristic crack width \( w_c \) had a similar affect on the load-displacement diagram of mixture S2P1. A ratio \( w_c \) to \( w_0 \) in the range of \( 1/60 \) resulted in a good fit.

### 11.1.5 Influence of the Equivalent Post-Cracking Tensile Strength

Figure 120 displays the effect of the equivalent post-cracking tensile strength on the load-displacement diagram of mixture S1P1. It was observed that the equivalent post-cracking strength had a pronounced affect on the tail on the curve. Kooiman proposed a ratio between the equivalent post-cracking tensile strength and the uniaxial tensile strength in the range of 0.2 to 0.3. A ratio of approximately 0.294 (\( 1/3.4 \)) resulted in a good fit.
The equivalent tensile post-cracking strength had a similar effect on the load-displacement diagram of mixture S2P1. A ratio in the range of 0.49 - 0.5 resulted in a good fit.

**11.2 Results and Discussion**

**Mixture S1P1**

**Parameters for the Bilinear Compression Stress-strain Relation**

A bilinear stress-strain relation was set for the compressive behaviour of SFRM mixture as shown in Figure 122. The linear elastic strain limit, as calculated in Subchapter 11.1.1, is equal to 2.7 %. The ultimate limit strain was set to 10 %. 

---

*Figure 120: Influence of the equivalent post cracking tensile strength on the load-displacement diagram of mixture S1P1*

*Figure 121: Influence of the equivalent post cracking tensile strength on the load-displacement diagram of mixture S2P1*
The ultimate strain limit does not have any influence on the flexural behaviour up to the considered displacement of 2 mm. The maximum compressive stress was never exceeded.

![Stress-strain behaviour in compression](image)

**Figure 122: Input for compressive stress-strain relations for SFRM [35]**

**Parameter for the uniaxial tensile strength**
Using the splitting tensile strength to calculate the uniaxial tensile strength led to a ratio of approximately 0.71. This resulted in an uniaxial tensile strength of approximately \( f_{ctm,ax} = 3.53 \text{ MPa} \). The elastic strain was recalculated in order that the tensile elastic modulus remained equal to the compressive elastic modulus. The elastic strain was recalculated as follows:

\[
\varepsilon_{c3} = \frac{f_{ctm,ax}}{E_t} = \frac{3.53}{23000} = 0.153\%_0
\]

**(11.3)**

**Parameters for the bilinear tensile softening relation**
The tensile softening relation is defined by four parameters or rather four degrees of freedom; the uniaxial tensile strength, the equivalent post-cracking tensile strength, critical and characteristic crack width. The uniaxial tensile strength was addressed in the previous chapter. The other parameters are discussed below.

*Equivalent Post-Cracking Tensile Strength*
The equivalent post-cracking tensile strength \( f_{ctm,eq,bil} \) is shown to be smaller than the uniaxial tensile strength \( f_{ctm,ax} \). Kooiman [35] discovered that the optimal post-cracking strength lies in the range of 0.2 to 0.3 multiplied by \( f_{ctm,ax} \) for SFRC. The results of the analysis led to a \( f_{ctm,eq,bil} \) to \( f_{ctm,ax} \) ratio in the order of 0.294 for mixture S1P1. This fits within the range that Kooiman proposed.

*Critical Crack Width*
The critical crack width \( w_0 \) cannot exceed the maximum embedded fibre length \( L_e \) for physical reasons. The maximum embedded length is equal to half the fibre length \( 1/2L_f = 6.5 \text{ mm} \). Kooiman proposed \( L_e \) to \( L_f \) ratio in range of 0.33 to 0.425. From the analysis a ratio in the order of 0.35 was obtained for mixture S1P1. This fits within the range that Kooiman proposed.
**Characteristic Crack Width**

Kooiman also proposed a characteristic crack width $w_c$ to critical crack width ratio in the range of $1/6 - 1/5$. However, the results of the analysis showed that a ratio in the range of $1/4$ was valid for mixture S1P1, which is much smaller (roughly 7 times smaller) than the ratios proposed by Kooiman. Possible reasons for this were specified in Subchapter Error! Reference source not found.. Furthermore, the fibre length is much shorter than the fibre length applied in Kooiman's research.

According to the aforementioned input values the following stress-strain/crack width relations were defined as shown in Figure 124 below.

![Figure 123: Tensile input relations for S1P1 [35]](image)

**Parameter for the Influence Length**

The influence length $l_i$ is a fictitious length parameter that defines a fracture zone of a finite length in the beam. The beam is considered to be infinitely stiff outside this region. The influence length is used to convert discrete displacements into compressive and tensile strains.

The influence length appeared to strongly influence the flexural peak load and elastic behaviour of the load-displacement curve. However, the tail of the load-displacement curve was hardly affected. This is explained by the fact that the influence length is directly related to the stiffness of the SFRM according to $\delta = \varepsilon \cdot l_i \rightarrow \varepsilon = \frac{\delta}{l_i} \rightarrow E = \frac{f}{\varepsilon} \cdot \frac{l_i}{\delta}$. Kooiman proposed an influence length of $l_i = 0.5 \times h_{le} = 62.5 \, \text{mm}$.

Based on the analysis this value appeared to be valid for mixture S1P1. Therefore, the influence length was set to 62.5 mm.

**Mixture S2P1**

**Parameters for the Bilinear Compression Stress-strain Relation**

The linear elastic strain limit is the same as for S1P1, i.e $\varepsilon_{le} = 2.7 \, \%$. The ultimate limit strain was also set to 10 %. 

**Parameter for the Uniaxial Tensile Strength**
The uniaxial tensile strength for S2P1 is assumed to be roughly in the same order as the tensile uniaxial strength for S1P1 as mentioned previously. Indeed, the analysis resulted in a value in the range of $f_{ctm,ax} = 3.5 - 3.6$ MPa for both mixtures.

**Parameters for the Bilinear Tensile Softening Relation**

*Equivalent Post-Cracking Tensile Strength*

The results of the analysis showed that the $f_{ctm,eq,bil}$ to $f_{ctm,ax}$ ratio is approximately 0.49 for mixture S2P1.

*Critical Crack Width*

Critical crack width $w_0$ for mixture S2P1 is about $0.33 \times L_i$.

*Characteristic Crack Width*

The analysis resulted in a ratio $w_c$ to $w_0$ equal to or smaller than 0.017, which is much smaller (roughly 10 times smaller) than the ratios proposed by Kooiman.

According to the aforementioned input values the following stress-strain/crack width relations for mixture S2P1 were defined as shown in Figure 124.

![Figure 124: Tensile input relations for S2P1 [35]](image)

*Parameter for the Influence Length*

The influence length remained the same for mixture S2P1, i.e. $l_i = 0.5 \times h_{ig} = 62.5$ mm

The parameters that define the stress-strain/crack width relation of the mixtures are summarized in Table 26.
Table 26: Total overview of the parameters of mixtures S1P1 and S2P1

<table>
<thead>
<tr>
<th></th>
<th>l₁ [mm]</th>
<th>ε_c3 [%]</th>
<th>ε_c3u [%]</th>
<th>f_ccm [MPa]</th>
<th>f_fctm,ax [MPa]</th>
<th>f_fctm,eq,bi [MPa]</th>
<th>ε_ct,ax [%]</th>
<th>w₀ [mm]</th>
<th>w_c [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1P1</td>
<td>62.5</td>
<td>2.70</td>
<td>10.00</td>
<td>61.91</td>
<td>3.53</td>
<td>1.04</td>
<td>0.154</td>
<td>4.55</td>
<td>0.11</td>
</tr>
<tr>
<td>S2P1</td>
<td>62.5</td>
<td>2.73</td>
<td>10.00</td>
<td>62.77</td>
<td>3.61</td>
<td>1.77</td>
<td>0.157</td>
<td>4.29</td>
<td>0.072</td>
</tr>
</tbody>
</table>

11.3 Concluding Remarks

In this chapter it was shown that through inverse analysis load-crack opening displacements diagrams could be accurately computed from the defined stress-strain/crack width relations that describe the compressive and tensile behaviour of the SFRM mixtures. In addition, the bilinear tensile softening response appears to be a good fit for describing the tensile post-cracking behaviour of the SFRM mixtures, while a simplified bilinear stress-strain relation describes the compression behaviour adequately.
12 Finite Element Analysis of SFRM

In Chapter 11 it was shown that a modified version of Hordijk’s multi-layer model can be used to determine a stress-strain/crack-width softening relation to define the tensile post-cracking behaviour of SFRM. Several material properties of the tensile softening stress-crack width relation were obtained with this model. In addition, the influence of these properties on the load-displacement diagrams was investigated.

Furthermore, in this part of the thesis the results of a study on the tensile post-cracking behaviour of SFRM by means of a finite element analysis are documented. It follows the preceding study of the tensile post-cracking behaviour by means of the multi-layer model. The parameters that were obtained from the multi-layer model were used as an input for the finite element model (FEM). These parameters were again optimized for the FEM. The optimized parameters that were obtained from the FEM were used in the study of a precast concrete profiled mortar joint. The latter is addressed in Part D – Finite Element Analysis of Precast Profiled Mortar Joint.

12.1 Finite Element Model

12.1.1 Finite Element Size & Type

A finite element analysis of the beam was conducted in order to verify the parameters that were obtained with the multi-layer model. The parameters verified in the finite element analysis were used for the study of a precast concrete profiled mortar joint. The behaviour of the beam was simulated by means of a 2D finite element model in ATENA. The model consists of finite elements that collectively define the geometry of the beam and allows for the finite element analysis to be executed. The size of the finite element may influence the accuracy of the results. Smaller elements may lead to a more accurate result. However, the calculation time increases greatly. ATENA documentation suggests at least 10 elements per plane in the Cartesian coordinate system. There are several different types of finite elements available in ATENA. These are divided into three groups:

- plane elements for 2D, 3D, and axi-symmetric analysis
- solid 3D elements
- special elements, which are used for modelling external cables, springs and gaps.

Plane 2D elements were applied in the finite element model, since a 2D model was considered to simulate the behaviour of the beam. Plane 2D elements can be subdivided into plane 2D quadrilateral elements and plane 2D triangular elements, see Figure 125. The effect of the type of element and the size of the quadrilateral elements were investigated. The results are documented in Chapter 12.2.
12.1.2 Material

ATENA offers a wide variety of material models that can be used to model brittle cementitious materials, such as concrete and FRCC. Material models like the standard SBETA material and the fracture-plastic materials (CC3DNon-linearCementitious, CC3DNon-linearCementitious2, CC3DNon-linearCementitious2User) are available. These models include the following of cementitious material behaviour:

- non-linear behaviour in compression including hardening and softening
- fracture of concrete in tension based on non-linear fracture mechanics
- bi-axial strength failure criterion
- reduction of compressive strength after cracking
- tension stiffening effect
- reduction of the shear stiffness after cracking (variable shear retention)
- two crack models: fixed crack direction and rotated crack direction.

The SBETA material model offers different options for the tensile softening diagram of cementitious materials. The tensile softening behaviour can be characterized by two different formulations for the crack opening:

- A fictitious crack model based on a crack-opening law and fracture energy, which is suitable for modelling of crack propagation in concrete. This was addressed in Chapter 9.
- A stress-strain relation, which is less suitable for crack propagation.

In order to utilize the parameters of the bilinear softening stress-strain/crack-width diagrams that were obtained with the multi-layer model, the fracture-plastic material model was required as opposed to the standard SBETA material model. The reason is that there are limitations in manipulating the input parameters of the stress-strain/crack-width relation of the SBETA material model. Many of the parameters cannot be directly adjusted as these are calculated automatically from integrated equations within ATENA in accordance with CEB-FIP Model Code 1990 and other building standards for concrete. Furthermore, a value for the fracture energy was required for the SBETA material model. However, this parameter was not calculated or measured in the preceding parts of this thesis. The ATENA documentation suggests in this case the material CC3DNon-linearCementitious2User fracture-plastic material model as an alternative. Moreover, it allows user-defined laws (relations) for selected materials, i.e. the entire stress-strain/crack-width relation in compression and tension can be defined.
See Subchapter 12.2 for a concise overview of the relevant material input parameters. Appendix H – FEA Input Parameters for SFRM displays additional input parameters that are valid for both mixtures.

12.1.3 Loads, Supports, Monitoring Points and Analysis Procedure

Loads and Supports
The load consisted of a prescribed deformation of approximately 0.02 mm per load step. Furthermore, the beam was supported on two rollers. The rigid horizontal displacement was restrained at the middle of the beam by the jack applying the force. See Figure 126 and Figure 127 for a schematization of the beam.

Monitoring Points
To measure the crack mouth-opening displacement (CMOD) during the load, monitoring points were placed on the left and right edge of the notch. The relative displacement was determined by subtracting the displacement on the left from the displacement on the right side of the notch.
Analysis Procedure

ATENA utilizes two basic methods to conduct finite element analyses. These methods are the Newton-Raphson and the Arc Length methods and can be used to solve non-linear equations. The Newton-Raphson method was applied as this is the most common and straightforward method. Newton-Raphson method uses the concept of incremental step by step analysis to obtain the following set of non-linear equations:

\[ K(p) \Delta p = q - f(p) \]  \hspace{1cm} (12.1)

Where

- \( q \) is the vector of total applied joint loads
- \( f(p) \) is the vector of internal joint forces
- \( \Delta p \) is the deformation increment due to loading increment
- \( p \) are the deformations of structure prior to load increment
- \( K(p) \) is the stiffness matrix relating loading increments to deformation increments
12.2 Results and Discussion

12.2.1 Effect of the Element Type, Lay-Out and Size

Three different finite element mesh lay-outs were applied and subsequently investigated, see Figure 130. Lay-out A consists of a full quadrilateral finite element mesh. Lay-out B consists of a full triangle finite element mesh. Lay-out C consists of a mix of both. It can be observed that in all three lay-outs there is a zone in the centre where the mesh size appears to be smaller. In the case of lay-out C, this so-called fracture zone was filled with triangle elements. The width of the fracture zone was assumed to be equal to the influence length \( l \), which was discussed in the previous section of this part of this thesis. See Chapter 11.
Figure 130: Finite element mesh; A quadrilateral, B triangle and C mixed

Mixture S2P1 and the parameters that were determined in this thesis were used as starting point for the investigation of the influence of the finite elements and of the lay-out. Figure 131 displays clearly that a full quadrilateral element mesh (solution A) led to a better fit with the experimental results. However, it was also observed that optimization of the parameters was required to get a better fit. For further analyses, a full quadrilateral mesh was applied.

Figure 131: Influence of element type and lay-out on Load-displacement diagram

ATENA suggests at least 6 to 10 elements per direction for qualitative and quantitative results, respectively. In the case of non-fracture zones, an element size of $a=0.015$ m was implemented. This resulted in 10 elements in the vertical direction. The element sizes in the fracture zone were, however, varied to investigate the effect of the element size. Four different element sizes were studied: $a=0.005$ m, $a=0.008$ m, $a=0.01$ m, $a=0.015$. The results are presented in Figure 132.
It can be clearly observed that when the element size is 0.005 m, the tale of the curve deviated significantly from the tale of the measured curve. Deviations up to 25% can be found within the considered displacements. In addition, the element size increased the computational time significantly compared to the larger element sizes. However, this element size led to a better fit with the measured curve around the peak load. Furthermore, the results show that applying larger element sizes (i.e. 0.008 m, 0.01 m and 0.015 m) resulted in a better fit regarding the tale of the curve. However, differences can be observed among the peak load of the different curves. Moreover, an element size of 0.008 m and 0.015 m led to an overestimation of the peak load whereas an element size of 0.01 m led to an underestimation of the peak load. For the sake of computational time and accuracy of the results, an element size of a=0.01 m was applied for further analyses. This resulted in 6 elements across the width of the fracture zone.

12.2.2 Elastic Modulus and Stiffness
Due to the investigation of the element type and size, it was observed that the previously calculated value for the E-modulus led to an inaccurate fit for the elastic part of the curve. Therefore, a parameter study on the flexural stiffness of beam was conducted in order to get the best fit. Many different parameters for the Modulus of Elasticity were investigated. Figure 112 displays the results of four different E-modulus values. It can be observed that the previously calculated value of $E_t=23000$ MPa resulted in an inaccurate fit for the elastic part of the curve. Interestingly, the interpolated value for the
E-modulus at 28 days $E_{t,28}=24870$ MPa, also did not result in the best fit. It was discovered that an E-modulus value above 30000 MPa resulted in the best fit. However, these values fell outside the range of the measured values as discussed in Subchapter 10.2, and are therefore improbable for the material under consideration. Moreover, these values are impractical when considering the age of the material.

![Load-Displacement diagram](image)

**Figure 134: Load-displacement diagram up a displacement of CMOD= 0.2 mm**

It can also be observed that the computed diagrams showed an early bend in the elastic part of the curve. Moreover, there was an early occurrence of an elasto-plastic phase before the peak. This resulted in differences between the computed and measured curve in the elastic phase. The reason for this is not clear. However, a possible explanation is that ATENA utilizes complex non-linear calculations with a margin for error in order to simulate the behaviour. The non-linearity of these calculations and margin for error could have led to early occurrence of the elasto-plastic phase. However, since the scale in which this occurs is very small, i.e. less than 5/100 mm, it is not considered significant for the total response of the beam. An E-modulus value of $E_t=24870$ MPa was applied in the analyses that followed.

### 12.2.3 Results of Analysis

A comprehensive and extensive parameter study was conducted for both mixtures. The results are summarized in this subchapter. Figure 135 and Figure 136 display the results of the simulations.
12.2.3.1 Mixture S1P1

Parameters for the Bilinear Compression Stress-Strain Relation

A bilinear stress-strain relation was assumed for the compressive behaviour of the SFRM mixture as shown in Figure 122. From the multi-layer model a value of 2.7 % was obtained for the linear elastic strain limit. However, the finite element analyses showed that a linear elastic strain limit in the range of 2 to 2.2 % resulted in a better fit. A value of 2.12 % was applied for further analyses. The ultimate limit strain was set to 10 %.

Like the multi-layer model, the ultimate strain limit did not have any influence on the flexural behaviour up to the considered displacement of 2 mm. The maximum compressive stress was never exceeded.
Parameter for the uniaxial tensile strength
From the multi-layer model a uniaxial tensile strength $f_{\text{fctm,ax}}$ of 3.53 MPa was obtained. The finite element analysis showed that values in the range of 3.5 to 3.7 led to a good result. A uniaxial strength of $f_{\text{fctm,ax}} = 3.66$ MPa was used for further analyses.

The strain at localization was calculated as follows:

$$\varepsilon_p = \frac{f_{\text{fctm,ax}}}{E_i}$$  \hspace{1cm} (12.2)

The analysis showed that a strain at localization in the range of 0.138 to 0.15‰ led to a good fit. A value of 0.147 ‰ was applied for further analyses.

Parameters for the bilinear tensile softening relation

Equivalent Post-Cracking Tensile Strength
A $f_{\text{fctm,eq,bil}}$ to $f_{\text{fctm,ax}}$ ratio of 0.29 was obtained from the multi-layer model. However, the finite element analysis showed that a ratio in the order of 0.27 resulted in a better fit for mixture S1P1. This fits within the range that Kooiman proposed.

Critical Crack Width
The critical crack width can be calculated as a percentage of the fibre length $L_f$. A ratio in the order of 0.35 was obtained from the multi-layer model. The critical crack width is therefore equal to $w_c = 0.35 \times L_f = 4.55$ mm. This value resulted in a good fit for the finite element model.

Characteristic Crack Width
A characteristic crack width $w_c$ to critical crack width ratio in the range of 0.024 was obtained from the multi-layer model and applied. However, the results of the finite element analysis showed that a ratio in the range of 0.029 led to a better fit for mixture S1P1.

According to the aforementioned input values the following stress-strain/crack width relations were defined as shown in Figure 138 below.
12.2.3.2 Mixture S2P1

Parameters for the Bilinear Compression Stress-strain Relation

A linear elastic strain limit of $\varepsilon_{c1} = 2.73 \%$ was obtained from the multi-layer model. However, the finite element analysis showed that a linear elastic strain limit in the range of 2.4 to 2.5 % led to a good fit. A value of 2.41 % was applied for further analyses. The ultimate limit strain was set to 10 %.

Parameter for the Uniaxial Tensile Strength

A uniaxial tensile strength $f_{ctm,ax}$ of 3.61 MPa was obtained from the multi-layer model. From the finite element model values in the range of 3.6 to 3.7 MPa were obtained. A uniaxial tensile strength $f_{ctm,ax}$ of 3.66 MPa was applied for further analyses.

The strain at localization was calculated as follows:

$$\varepsilon_p = \frac{f_{ctm,ax}}{E_i}$$  \hspace{1cm} (12.3)

As with mixture S1P1, the analysis showed that a strain at localization in the range of 0.139 to 0.145\% led to a good fit. A value of 0.145 % was applied for further analyses.

Parameters for the Bilinear Tensile Softening Relation

Equivalent Post-Cracking Tensile Strength

The results of the analysis showed that the $f_{ctm,eq,bil}$ to $f_{ctm,ax}$ ratio around 0.49 led to a good fit for mixture S2P1. Moreover, the equivalent tensile post-cracking strength is equal to $f_{ctm,eq,bil} = 1.79$ MPa.

Critical Crack Width

A critical crack width to fibre length ratio in the order of 0.33 was obtained from the finite element model. This value is equal to the value obtained from the multi-layer model. The critical crack width can therefore be calculated as $w_0 = 0.33 \times 13 = 4.29$ mm.

Characteristic Crack Width

The analysis resulted in a $w_c$ to $w_0$ ratio equal to or smaller than 0.017. This range is smaller than the range obtained from the multi-layer model.
According to the aforementioned input values the following stress-strain/crack width relations for mixture S2P1 were defined as shown in Figure 124.

\begin{align*}
\sigma_{\text{ct}} / \sigma_{\text{fem,ax}} &= 1.0 \\
\varepsilon_{\text{ct}} / \varepsilon_{\text{fem,ax}} &= 0.145 \\
\sigma_{\text{f}} / \sigma_{\text{fem,ax}} &= 1.0 \\
\varepsilon_{\text{f}} / \varepsilon_{\text{fem,ax}} &= 0.017
\end{align*}

Figure 139: Tensile input relations for S2P1 [35]

12.2.4 Crack Formation

Figure 140 shows the typical crack formation of the beam specimen after testing and analysis, respectively.

12.2.5 Total Overview of the Parameters of Mixtures S1P1 and S2P1

The optimized input parameters of the basic properties, tensile softening and compressive softening stress-(fracture) strain diagrams of the CC3DNonLinCementitious2User material for both mixtures are listed below:
Table 27: Input parameters for mixture S1P1

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic properties</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$E_c$</td>
<td>MPa</td>
<td>24870</td>
<td>Initial elastic modulus valid for compression and tension</td>
</tr>
<tr>
<td>$\nu$</td>
<td>-</td>
<td>0.2</td>
<td>Poisson ratio</td>
</tr>
<tr>
<td>$f_t$</td>
<td>MPa</td>
<td>3.66</td>
<td>Tensile (peak) strength</td>
</tr>
<tr>
<td>$f_c$</td>
<td>MPa</td>
<td>-52.62</td>
<td>Compressive cylinder strength; $f_c = 0.85 \times f_{ctm}$</td>
</tr>
<tr>
<td>Tensile properties (tensile softening)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\varepsilon_1$</td>
<td>-</td>
<td>0</td>
<td>Fracture strain at tensile strength</td>
</tr>
<tr>
<td>$\sigma_1/f_t$</td>
<td>-</td>
<td>1</td>
<td>Ratio stress to tensile strength at corresponding fracture strain</td>
</tr>
<tr>
<td>$\varepsilon_2$</td>
<td>-</td>
<td>2.08E-03</td>
<td>Fracture strain corresponding to $w_c$; $\varepsilon_2 = w_c/l_i$, $l_i = l_{ch}$</td>
</tr>
<tr>
<td>$\sigma_2/f_t$</td>
<td>-</td>
<td>0.27</td>
<td>Ratio stress to tensile strength; $f_{fctm,eq,bil}/f_{ctm,ax}$</td>
</tr>
<tr>
<td>$\varepsilon_3$</td>
<td>-</td>
<td>7.28E-02</td>
<td>Fracture strain corresponding to $w_0$; $\varepsilon_3 = w_0/l_i$, $l_i = l_{ch}$</td>
</tr>
<tr>
<td>$\sigma_3/f_t$</td>
<td>-</td>
<td>0</td>
<td>Ratio stress to tensile strength; $f_{fctm,eq,bil}/f_{ctm,ax}$</td>
</tr>
<tr>
<td>$l_{ch}$</td>
<td>m</td>
<td>6.25E-02</td>
<td>Characteristic size is assumed equal to influence length; $l_{ch} = l_i$</td>
</tr>
<tr>
<td>$\varepsilon_p$</td>
<td>-</td>
<td>1.47E-04</td>
<td>Strain after which softening becomes localized; $\varepsilon_p = f_t/E_c$</td>
</tr>
<tr>
<td>Compressive properties (compressive softening)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\varepsilon_1$</td>
<td>-</td>
<td>-1.00E-02</td>
<td>Ultimate fracture strain; $\varepsilon_1 = \varepsilon_{c3u}$</td>
</tr>
<tr>
<td>$\sigma_1/f_c$</td>
<td>-</td>
<td>0</td>
<td>Ratio stress to compressive strength at corresponding fracture strain</td>
</tr>
<tr>
<td>$\varepsilon_2$</td>
<td>-</td>
<td>-2.12E-03</td>
<td>Fracture strain corresponding to $\varepsilon_{c3}$; $\varepsilon_2 = \varepsilon_{c3} = \varepsilon_p$, $l_i = l_{ch}$</td>
</tr>
<tr>
<td>$\sigma_2/f_c$</td>
<td>-</td>
<td>1</td>
<td>Ratio stress to compressive strength; $\sigma = f_c$</td>
</tr>
<tr>
<td>$\varepsilon_3$</td>
<td>-</td>
<td>0.00E+00</td>
<td></td>
</tr>
<tr>
<td>$\sigma_3/f_c$</td>
<td>-</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>$l_{ch}$</td>
<td>m</td>
<td>6.25E-02</td>
<td>Characteristic size is assumed equal to influence length; $l_{ch} = l_i$</td>
</tr>
<tr>
<td>$\varepsilon_p$</td>
<td>-</td>
<td>-2.12E-03</td>
<td>Strain after which softening becomes localized; $\varepsilon_p = f_t/E_c$</td>
</tr>
</tbody>
</table>
Table 28: Input parameters for mixture S2P1

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Basic properties</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( E_c )</td>
<td>MPa</td>
<td>24870</td>
<td>Initial elastic modulus valid for compression and tension</td>
</tr>
<tr>
<td>( \nu )</td>
<td></td>
<td>0.2</td>
<td>Poisson ratio</td>
</tr>
<tr>
<td>( f_t )</td>
<td>MPa</td>
<td>3.66</td>
<td>Tensile (peak) strength</td>
</tr>
<tr>
<td>( f_c )</td>
<td>MPa</td>
<td>-53.35</td>
<td>Compressive (elastic) strength; ( f_c = 0.85 \times f_{cm} )</td>
</tr>
</tbody>
</table>

| **Tensile properties (tensile softening)** |      |       |                                                           |
| \( \varepsilon_1 \) |       | 0     | Fracture strain at tensile strength                       |
| \( \sigma_1/f_t \) |       | 1     | Ratio stress to tensile strength at corresponding fracture strain |
| \( \varepsilon_2 \) |       | 1.15E-03| Fracture strain corresponding to \( w_c \); \( \varepsilon_2 = w_c/l_i, l_i = l_{ch} \) |
| \( \sigma_2/f_t \) |       | 0.49  | Ratio stress to tensile strength; \( f_{fctm,eq,bil}/f_{ctm,ax} \) |
| \( \varepsilon_3 \) |       | 6.86E-02| Fracture strain corresponding to \( w_0; \varepsilon_3 = w_0/l_i, l_i = l_{ch} \) |
| \( \sigma_3/f_t \) |       | 0     | Ratio stress to tensile strength; \( f_{fctm,eq,bil}/f_{ctm,ax} \) |
| \( l_{ch} \)   | m    | 6.25E-02| Characteristic size is assumed equal to influence length; \( l_{ch} = l_i \) |
| \( \varepsilon_p \) |       | -1.45E-04| Strain after which softening becomes localized; \( \varepsilon_p = f_c/E_c \) |

| **Compressive properties (compressive softening)** |      |       |                                                           |
| \( \varepsilon_1 \) |       | -1.00E-02| Ultimate fracture strain; \( \varepsilon_1 = \varepsilon_{ctu} \) |
| \( \sigma_1/f_c \) |       | 0     | Ratio stress to compressive strength at corresponding fracture strain |
| \( \varepsilon_2 \) |       | -2.41E-03| Fracture strain corresponding to \( \varepsilon_{ct}; \varepsilon_2 = \varepsilon_{ct} = \varepsilon_p, l_i = l_{ch} \) |
| \( \sigma_2/f_c \) |       | 1     | Ratio stress to compressive strength; \( \sigma = f_c \) |
| \( \varepsilon_3 \) |       | 0.00E+00|                                                      |
| \( \sigma_3/f_c \) |       | 0     |                                                      |
| \( l_{ch} \)   | m    | 6.25E-02| Characteristic size is assumed equal to influence length; \( l_{ch} = l_i \) |
| \( \varepsilon_p \) |       | -2.12E-03| Strain after which softening becomes localized; \( \varepsilon_p = f_c/E_c \) |

Appendix H – FEA Input Parameters for SFRM displays additional material input parameters that are required for the user material.

### 12.3 Concluding Remarks

In this chapter it was shown that a finite element model can be used to simulate the behaviour of SFRM beam specimens. Stress-strain laws for tension and compression were defined for a user material that represented SFRM mixtures. From the analyses that were carried out, load-crack opening displacement diagrams were computed. The parameters of the stress-strain relation were optimized to get the best fit.
The sizes of the finite elements and two different types of finite elements were investigated, namely the quadrilateral and triangle finite elements. It was shown that the quadrilateral elements resulted in the best fit. Furthermore, it was observed that if the size of the finite elements is chosen small (i.e. $a=0.005$ m), the tale of the curve that represents the post-cracking behaviour deviated significantly from the measured curve. In contrast, if the size of the finite elements is chosen too large (i.e. $a=0.01$ m and $a=0.015$) finite elements led to an inaccurate representation of the results. Lastly, the response of the material in the elastic phase indicated that the initially calculated elastic modulus may have been an underestimation of the true elastic modulus. However, it is assumed that the deviation may be the result of the non-linear response of the material as a result of the non-linear finite element analyses that are carried out with ATENA.
PART D
FINITE ELEMENT ANALYSIS OF PRECAST PROFILED MORTAR JOINT
13 Introduction

The final objective of this thesis was to investigate the effect of the steel fibre reinforced mortar on a precast concrete profiled mortar joint. This was done by means of a finite element analysis. The results of the analysis were compared with the results of an experimental investigation of high quality mortar joints currently being conducted by Van Keulen [2]. A comparison is made between three mortar mixtures, i.e. plane TiksoMortar K70, SFRM with 30 kg/m$^3$ and 60 kg/m$^3$ of steel fibres. The optimized parameters that were obtained from the FEA of the beam specimen were used as input for the user defined material of the mortar in the joint for FEA.

This part of the report is structured as follows. In Chapter 14, the material, functional design of the precast mortar joint and load are addressed. Chapter 15 addresses the finite element model. Chapter 16 addresses the finite element analyses and the results.
14 Precast Concrete Profiled Mortar Joint

14.1 Profiled Joint

In Chapter 5, two different types of profiled mortar joints were addressed, i.e. equal and shifted profiled mortar joints. Van Keulen’s research showed that the shifted profiled mortar joints have a higher load bearing capacity when compared to the equal profiled mortar joints. Moreover, in similar conditions, the load bearing capacity of the shifted profiled mortar joint was approximately 68% higher. However, the stiffness of the joints appeared to be in the same order. Because of this, the shifted profiled mortar joint was therefore considered for the finite element analysis in order to investigate the effect of the steel fibre reinforced mortar mixtures on the joint.

![Figure 141: Profiled Mortar Joints with scheme of struts; equal profile (left) and shifted profile (right) [2]](image)

14.2 Functional Design of the Joint

As mentioned in Subchapter 5.1.2, L-shaped elements were used as specimens in the experimental investigation of the profiled mortar joints. The dimensions are displayed in Figure 142. The joints are 200 mm deep. Figure 158 also shows the dimensions of the shifted profile. The width of the joint is 75 mm (3x25). The dimensions were determined based on literature review and preliminary research that was conducted by Van Keulen.

![Figure 142: Dimensions of L-shape element and shifted profile [2]](image)
14.3 Material
The entire joint consists of several different materials. These are as follows:
- Reinforced precast elements: concrete quality C53/65
- Mortar: Cuglaton Tiksomortar K70
- SFRM
  - Mixture S1P1 (with 30 kg/m$^3$ of fibres)
  - Mixture S2P1 (with 60 kg/m$^3$ of fibres)
- Threaded steel rods S235 (M24)

Since, the flexural behaviour and, subsequently, the uniaxial tensile strength of the tiksomortar K70 was not determined, the parameters were estimated based on parameters of the SFRM. Gopalaratnam et al. [52] showed in their investigation of the tensile failure of the SFRM that under similar conditions the SFRM boasted a 7% higher uniaxial tensile strength compared to the plane mortar. In addition, the experimental results of the flexural behaviour of the beam specimens showed no increase in flexural peak strength with two times as much fibres. Therefore, the uniaxial tensile strength of the plane mortar was assumed to be equal to or no more than 7% lower than the uniaxial strength of the mixture S1P1. The same is valid for the compressive strength. Furthermore, the elastic modulus was assumed to be the same for both mixtures. The estimated material properties of the Tiksomortar K70 are displayed in Appendix I – FEA Input Parameters for Profiled Mortar Joint.

14.4 Loads
A shear load is applied on the structure. This load is generated by the testing machine, see Figure 33. This load represents the shear forces caused by lateral loads on precast concrete shear wall structures or concrete cores. As a result of the compression struts that arise in the shear keys, horizontal thrust forces will occur, see Figure 141. These forces are resisted by the surrounding structural elements. The magnitude of the reaction forces depends on the stiffness of the elements surrounding the joint. In order to apply similar conditions in a laboratory setting, four threaded steel rods are attached to the elements, see Figure 33. Two rods are placed at the top and bottom, respectively. The steel rods will provide a stiffness to the joint. In addition, when the joint is loaded by a shear force, the threaded steel rods will provide a counter force. This results in a higher load bearing capacity for the joint. Steel rods with a higher stiffness will provide a larger counter force. Furthermore, the steel rods have the purpose of holding both elements in place.
Pre-tensioning is not commonly applied in vertical precast concrete profiled mortar joints. Therefore, pre-tensioning was not considered for the analysis. However, since the steel rods have the purpose of keeping the elements together, a small pre-tensioning of 0.1 N/mm$^2$ was applied.
15 Finite Element Model

A finite element analysis of the joint was conducted in order to investigate the effect of the SFRM on the load bearing capacity and stiffness of the joint. The finite element model that was used in this analysis was initially developed by Van Keulen and was used in his analysis of the shear behaviour of the profiled joints. The parameters that were obtained from the FEA of the beam specimen were used as input for the user material of the SFRM in the finite element model of the joint. Like the FEA of the beam, the finite element analysis program ATENA 2D was used to conduct a FEA of the joint.

In this chapter the aspects of the finite element model are expanded on. First, the finite element type that is used to model the geometry of the joint is discussed, followed by the element type and sizes. The material models that were used to model the precast elements and the joint are addressed in Subchapter 12.1.2. Subsequently, the loads, supports and monitoring points are discussed in the subchapters that follow. This chapter concludes with a description of the analysis procedure in Subchapter 0.

15.1 Finite Element Size & Type

Plane 2D quadrilateral elements were used in the finite element model, see Figure 143.

![Figure 143: Geometry of quadrilateral CCIsoQuad (ATENA theory)](image)

Several different sizes of the quadrilateral elements were applied in the structure, see Figure 144. As shown in Subchapter Results and Discussion12.2, the size of the element affects the outcome of the results. Therefore, an element size of 0.01 m was applied for the mortar. This size was intentionally chosen in order to keep the size of the finite element of the mortar in the joint the same as the size of the finite elements in the fracture zone in the model of the beam specimen. This way the size effect of the elements can be neglected. Furthermore, a so-called 'transition' or 'fracture' zone was defined around the mortar. A slightly larger element size of 0.0125 was applied in this zone in order to decrease the amount of elements. This decreases the analysis run time. Lastly, an element size of 0.05 m was implemented for the precast concrete elements.
15.2 Material

As mentioned in Chapter 12.1, ATENA offers a wide variety of material models that can be used to model brittle cementitious materials. For the finite element model under consideration, two different materials were applied, i.e. SBETA material for the precast concrete elements and the fracture-plastic materials (CC3DNon-linearCementitious2User) for the mortar. Other materials that were defined:

- Steel Reinforcement S235
- Bond for Reinforcement
- 2D interface: interface between concrete element and mortar. The cohesion, friction coefficients and stiffness can be defined.
- Steel Supports: plane stress elastic isotropic
- External cable for pre-stressing: plane stress elastic isotropic

The parameters of the material properties are listed in Appendix I – FEA Input Parameters for Profiled Mortar Joint.
15.3 Loads, Monitoring Points and Analysis Procedure

**Loads**
The load consisted of a prescribed deformation of approximately 0.1 mm per load step. A small pre-tensioning force was applied as mentioned in Subchapter 14.4.

**Monitoring points**
Monitoring points were placed in order to measure the horizontal and vertical displacements at the top, middle and bottom section of the joint on both sides. In case of the horizontal displacements, the relative displacement was determined by subtracting the displacement on the left from the displacement on the right.
Analysis Procedure

The Newton-Raphson method was applied as this is the most common and straightforward method. This method was described in the previous part of this thesis.
16 Results and Discussion

16.1 Results

Many simulations were conducted in order to optimize the parameters and to obtain more accurate diagrams. The final results are presented in the figures below. Figure 147 displays the results of the finite element analyses of the profiled mortar joint with three different mixtures. The maximum shear strength are summed up below.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Shear Strength [kN]</th>
<th>Strength Increase [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mortar K70</td>
<td>680</td>
<td>0</td>
</tr>
<tr>
<td>Mixture S1P1</td>
<td>788</td>
<td>16%</td>
</tr>
<tr>
<td>Mixture S2P1</td>
<td>792</td>
<td>16%</td>
</tr>
</tbody>
</table>

It can be observed that the SFRM mixtures led to a higher shear strength compared to the plane mortar. However, it can also be observed that both SFRM mixtures led to approximately the same result. Moreover, the shear strength remained more or less the same. The post-peak strength differs, though marginal. It was discovered that the tensile strength and, particularly, the compressive strength of the mortar mostly affects the shear strength of the joint. Since both mixtures possessed a uniaxial tensile and compressive strength in the same order, the shear strength of the joint hardly increased.

![FEA load-displacement diagram](image-url)

Figure 147: FEA load-displacement diagram of mixtures
16.2 Finite Element Analysis versus Experimental Results

When the results of the experiments of the profiled mortar joint with plane mortar are compared with the results of the finite element analyses, it can be observed that there is a large deviation of more than 70% in the pre-peak behaviour of the joint. A snap can be observed in the beginning of the diagram, see Figure 151. It is argued that up to this point the shear load is predominantly transferred through cohesion. At the point of the snap, the cohesion strength is then surpassed and the load is then transferred through compressive diagonal struts and friction. Many optimization and simulations were conducted in order to obtain this behaviour, however to no avail. It was therefore concluded that the ATENA Model cannot simulate this effect. However, the peak load and the ductility of the joint are in relatively good agreement with the experimental results. The maximum deviation in peak load between the computed and measured curves is 17%.

Other factors that could lead to differences between the finite element analysis and the experimental results are mentioned below:
- Shrinkage of the mortar was observed in the joints. Shrinkage of the mortar may influence the load-bearing capacity. The larger the shrinkage the lower the load-bearing capacity will be.
- The finite element model assumes an ideal situation for the joint, whereas in practice there can be deviations in the stiffness, geometry of the profiles, imperfections, and the degree of filling of the mortar.

![FEA load-displacement diagram](image1)

**Figure 150:** load-displacement diagram; Mortar K70 FEA vs experiment

![FEA load-displacement diagram](image2)

**Figure 151:** load-displacement diagram; snap in the measured curves is visible.

Furthermore, it is worth mentioning that the mortar’s properties that were obtained through the previous finite element analyses concerns the SFRM with an age of 7 to 8 days. However, the experiments of the precast joints were carried out after the mortar reached an age of 28 days or more. At 28 days, the mortar would have an increased tensile strength compared to the mortar with an age of 7 days. The increased tensile strength would result in higher shear and post-racking strength for the
finite element model of the joint. Consequently, the deviation between the measured and computed curves would increase.

16.3 Crack Formation

16.3.1 Reference Mortar

Figure 152 displays the stages of crack formation. It can be observed that the cracking first appears in the concrete elements.

- Stage 1:
  - A: In the beginning stages of the analysis, cracking occurs in the smallest cross section of the profiled joint. These appear to be caused by tensile stresses.
  - B: The concrete elements experience early cracking.

- Stage 2:
  - C: Cracking in the mortar increases. These cracks are caused by tensile stresses.
  - D: Shear cracks appear
  - Cracks in the concrete elements increase.

- Stage 3: Diagonal cracks start to take form.

Figure 152: Stages of crack formation

Figure 153 and Figure 154 show the crack formation of the specimens versus the crack formation in the finite element model. The crack formation of the finite element model shows some similarities with the tested specimens.

- 1: Cracking along the surface
- 2: Cracking in the concrete element. The cracks are exaggerated in the finite element model.
- 3: Shear cracking
- 4: Cracking of the concrete elements
- 5: Cracking along the surface

Figure 153: Crack formation of shifted profiled mortar joint vs. FEM; specimen 2 phase 2
16.3.2 Fibre Reinforced Mortar

Figure 155 and Figure 156 display the crack formation of the finite element model of mixtures S1P1 and S2P1. The crack formation is largely similar to the crack formation of the reference material. However, it can be observed that much more cracks can be found in the concrete elements between the second and fifth shear key.
Figure 155: Stages of crack formation mixture S1P1

Figure 156: Crack formation mixture S2P1
16.4 Concluding Remarks

The precast concrete profiled mortar joint was modelled and analysed. It was shown that the results of
the finite element model deviated from the results of the experiments. The difference is caused by a
snap which can be observed in the experimental results. This snap was reproducible in the finite
element model. This snap is most likely caused by failure of the cohesion strength at the interface
between the concrete element and mortar. The stiffness, however, appears to be in the same order.
Furthermore, the crack formation showed some similarities between the FEA and the experiments.
Furthermore, more cracks appear in the concrete elements when fibre reinforced mortar is applied.
PART E
CONCLUSIONS &
RECOMMENDATIONS
17 Conclusions

Introducing fibres into the mortar mix of profiled mortar joints is a practical method of improving the shear strength of such joints. This project was undertaken in order to study the feasibility of such an approach. This thesis comprises the results of an investigation into the mechanical properties of the fibre reinforced mortar and an analysis of the fibre reinforced profiled mortar joint. First, a literature review was conducted in order to study the basics of the fibres, profiled mortar joints and fibre reinforced composites. Subsequently, different analysis methods were applied in order to investigate the properties and behaviour of the material and the behaviour of the joint. These methods were the following:

- Inverse analysis approach in order to obtain basic parameters that could be used as starting point for the FEA.
- FEA of the beam specimen in order to optimize the obtained parameters and to identify other relevant parameters that can be used in the analysis of the joint.
- To analyse the joint structure with the different mixtures by means of finite element analysis using the input parameters obtained from the FEA of the beam specimens.

Based on the result the following research question was answered:

“How does fibre reinforced profiled mortar joints behave in terms of strength and stiffness in precast concrete shear wall structures?”

From the analyses of the results the following conclusions can be drawn:

Experiments Phase I/II/III:

- Shrinkage
  The fibres appear to have no significant influence on reducing the shrinkage of the thixotropic mortar. The water dosage, however, plays a larger role on the magnitude of the drying shrinkage. The higher the water dosage the higher the shrinkage will be.

- Compression Strength
  The compression strength of the mixtures was positively influenced by the presence of fibres. An increase of 10 - 20% was observed for the considered fibre dosages at mixtures ages from 7 to 28 days, respectively. However, in the case of larger specimens, it appeared that both PVA and steel fibres may have a minimal effect in increasing the compression strength at the considered fibre dosages. The results also showed that the mortar pump has a positive influence on the compression strength of the mixtures, though marginal. Higher steel fibre dosages resulted in a higher compression strength. A maximum increase of 15% was observed for the considered fibre dosages.

- Flexural Strength
  The flexural strength was also significantly and positively influenced by the presence of the fibres. Increases of more than 20% were observed. In addition, it was observed that in the case of the larger specimens, the PVA fibres appeared to have contributed more to the limit of proportionality (LOP) at the fibre dosage of 30 kg/m³ as opposed to 60 kg/m³. Moreover, a ratio of increase in LOP between
mixtures S1 and S1P1 of 18% was observed as opposed to 4.7% between mixtures S2 and S2P1. The same is valid for the residual flexural strength.

The results provided compelling evidence that the flexural tensile strength is negatively influenced by the mortar pump. Specimens containing 30 kg/m$^3$ of steel fibres had a 7% lower flexural strength compared to the reference mortar. In addition, specimens containing 60 kg/m$^3$ of steel fibres had a 23% lower flexural strength. Furthermore, the PVA fibres appeared to have had no effect on the flexural strength at the fibre dosage under consideration, despite pumping. The decrease in strength was attributed to the subpar fibre distribution and orientation caused by pumping the mortar.

- Splitting Tensile Strength

It was observed that the splitting tensile strength is significantly influenced by the presence of fibres. An increase in splitting tensile strength of approximately 14% was observed for mixture S2 when compared with mixture S1. In the case of mixture S2P1, an increase in splitting tensile strength of 20% was observed when compared with mixture S1. The steel fibres appear to have played a larger role in increasing the strength as opposed to the PVA fibres.

Modelling behaviour of SFRM by means of inverse analysis

It was shown that an inverse analysis can be used to analyse the behaviour of SFRM. Load-crack opening displacements could be accurately computed from defined stress-strain/crack-width relations that describe the compressive and tensile behaviour of SFRM mixtures. A bilinear compressive and tensile softening response appears to be a good fit for describing the tensile post-cracking and compressive behaviour of the SFRM. From the results parameters were obtained that were used in the finite element analysis of the beam specimens.

Finite element analysis of the beam specimen

From the finite element analysis of the beam specimens the following conclusions were drawn:

It was shown that the finite element model can be used to simulate the behaviour of SFRM beam specimens. Stress-strain laws for tension and compression were defined for a user material that represented the SFRM mixtures. The parameters that were obtained from the inverse analysis were optimized in order to get the best fit. These optimized parameters were then used in the analyses of the profiled mortar joint.

It was observed that the finite element sizes and type may influence the accuracy of the results. In terms of finite element type, the quadrilateral finite elements resulted in the best fit as opposed to the triangular finite elements. Furthermore, four different element sizes were investigated, i.e. 0.005 m, 0.008 m, 0.01 m and 0.015 m. It was observed for the small finite element size (i.e. a=0.005 m) that the tale of the curve that represents the post-cracking behaviour deviated significantly from the measured curve. Deviations up to 25 % can be found within the maximum considered CMOD of 2 mm. In contrast, an inaccurate representation of the results was observed for the larger element sizes (i.e. a=0.01 m and a=0.015). A deviation in the elastic part of the curve can be also observed. It assumed that this is caused by the non-linear response of the material as a result of the non-linear finite element analyses that are carried out by ATENA 2D.
Finite element analysis of the joint

The precast concrete profiled mortar joint was modelled and analysed. It was observed that the results of the analysis deviated from the results of the experiments. Large deviations ranging from 70 to 80% were observed. The deviations were caused by a snap which can be observed in the experimental results. This snap was not reproducible in the finite element model. The snap is likely the result of failure of the cohesion at the interface between the concrete element and mortar. On the other hand, the peak load and the ductility of the joint were in relatively good agreement with the experimental results. The maximum deviation in peak load between the computed and measured curves was 17%. Lastly, the crack formation that was observed from the experiments and the FEA showed some similarities.
18 Recommendations

The following recommendations can be given:

Experiments
In this thesis only one specific type of PVA fibre was used. The purpose of adding the PVA fibres was to assist in reducing shrinkage of the mortar. However, the results showed clearly that the PVA fibres had little to no affect on the shrinkage. In addition, the strength properties were also not affected by the PVA fibres. Other PVA fibres are on the market with improved physical and material properties that may have had a better effect.

Adding fibres to the mix resulted in an improved performance of the mixtures. Mainly the post-cracking behaviour was positively affected. The results of the experiments were very promising. However, in the third phase of testing, the effect of the execution process on the mechanical properties was clearly observed. It was shown that the mortar pump had a negative effect on the flexural strength of the fibre reinforced mortar. It is hypothesized that due to differences in the diameter of the hose of the pump, accelerations and pressure differences in the hose, separation of the mortar and fibres occur. The separation of fibres and mortar may also be enhanced by the differences in density and aggregate size. Therefore, it is recommended that a study is conducted on the pump and improving the flow of the fibre reinforced mortar in order to reduce distortion of the fibre distribution.

FEA of the joint
The FEA of the joint showed that SFRM can improve the shear capacity of the joint. However, the FEA assumes an ideal situation, whereas in practice there are many factors that may influence the shear capacity of the joint. The experimental results showed clearly in all three specimens that there is a moment where a snap occurs before the shear continues increasing unto the peak strength. It is assumed that the snap occurs once the cohesion strength is surpassed. ATENA 2D provides the possibility to introduce cohesion and friction through a 2D interface. Due to time constraint, the 2D interface could not be fully investigated. This requires further research in order to improve the finite element model.
Bibliography

[12] A. Lambrechts, "Steel – and Synthetic Fibre Reinforced Concrete
Which fibre to use for which application and why?," in Dramix, ed: Bekaert, 2011.


Standards and Recommendations

**Fibres**
- **NEN-EN 14889-1** Fibres for concrete – Part 1: Steel fibres – Definitions, specifications and conformity
- **NEN-EN 14889-2** Fibres for concrete – Part 2: Polymer fibres – Definitions, specifications and conformity

**Test Methods**
- **NEN-EN 14845-1** Test methods for fibres in concrete – Part 1: reference concretes
- **NEN-EN 14845-2** Test methods for fibres in concrete – Part 2: Effect on concrete
- **NEN-EN 14651** Test method for metallic fibered concrete – Measuring the flexural tensile strength (limit or proportionality (LOP), residual)
- **NEN-EN 12390-3** Beproeving van verhard beton – Deel 3: Druksterkte van proefstukken
- **NEN-EN 12390-6** Beproeving van verhard beton – Deel 6: Splijttreksterkte van proefstukken

**Mortar**
- **CUR-Aanbeveling 24** Krimparme cementgebonden mortels
CUR-Aanbeveling 108

Ontwerp en uitvoering van mortelvoegen in prefab betonconstructies
APPENDICES
Cementgebonden

CUGLATON® TIKSMORTEL K70

CUGLATON TIKSMORTEL K70 is een krimparme cementgebonden mortel met thixotrope eigenschappen, speciaal ontwikkeld voor het monteren van prefab montagesystemen, zonder gebruik van bekisting. CUGLATON TIKSMORTEL K70 heeft een uitstekende verpompbaarheid, stabiliteit en hechtkracht. CUGLATON TIKSMORTEL K70 kan worden toegepast in laagten tot ca. 60 mm.

Toepassingsgebied

- Vullen van sluiuimen onder prefab betonnen elementen d.m.v. pomp en platen van prefab betonnen elementen in een speciebed
- Vullen van verticale voegen tussen prefab betonnen elementen
- Horizontaal vervijlen van ankers
- Vullen van kraanhanen onder helling
- Horizontaal vervijlen van ankers
- Vullen van verticale voegen tussen prefab betonnen elementen

Classificatie volgens CUR-Aanbeveling 24

KRIMPARME CEMENTGEBONDEN MORTELS

<table>
<thead>
<tr>
<th>Materiaalsoort</th>
<th>Troffenmortel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Morteltype</td>
<td>0,5 mm</td>
</tr>
<tr>
<td>Sterkteklasse</td>
<td>K70</td>
</tr>
<tr>
<td>Milieuklasse</td>
<td>X0 /m/XA3</td>
</tr>
<tr>
<td>Zwellings</td>
<td>&gt; 0,1% &lt; 2,0 %</td>
</tr>
<tr>
<td>Gemiddelde krimp</td>
<td>&lt; 0,75 mm/min</td>
</tr>
</tbody>
</table>

Gebruiksaanwijzing

Voorbehandeling

Het oppervlak waarop CUGLATON TIKSMORTEL K70 moet worden aangebracht dient schoon te zijn, cementhuid verwijderen en er mag geen vij water aanwezig zijn.
- Voorbehandeling met water.
  - Het oppervlak dient alleen voorbevochtigd te worden met water (Let op: geen vrij water).
- Voorbehandeling met CUGLACRETE HECHTPRIMER CEMENTGEBONDEN, een cementgebonden polymersmodificeerd systeem, of CUGLACRETE HECHTPRIMER EPOXY SEALER, een systeem op epoxy basis.

Mengen

CUGLATON TIKSMORTEL K70 machinaal mengen tot een homogene mengsel. Mengtijd afhankelijk van het type menger, ca. 3 minuten.

Waterdosering

Waterbandbreedte: 3,7 – 4,2 ltr/25 kg mortel. Doezeer, binnen de aangegeven waterbandbreedte, zodanig veel water dat een mortelplas met een spreidmaat van ca. 140 mm wordt bereikt.

Nabehandeling

Het afgewerkte vlak moet zorgvuldig tegen uitdrogen worden beschermde met CUGLA CURING COMPOUND, of afdekken met plastic folie.

Opslag en houdbaarheid

Indien droog opgeslagen, houdbaar tot 6 maanden na productiedatum, zoals vermeld op de verpakking.
Gezondheid en milieueaspecten

Cugla adviseert om tijdens gebruik van CUGLATON:
• Geschikte handenchoenen en veiligheidsbril te dragen.
• Contact met ogen en huid te vermijden.
• Indien het product in de ogen komt onmiddellijk te spoelen met water en medisch advies in te winnen.
• Bij inslikken onmiddellijk een arts te raadplegen en de verpakking of het veiligheidsinformatieblad te tonen.

Technische gegevens bij 20°C/65% r.v.

<table>
<thead>
<tr>
<th>Maximale korrel</th>
<th>0,3 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cementsoort</td>
<td>Portland</td>
</tr>
<tr>
<td>Waterbandbreedte</td>
<td>3,7 – 4,2 ltr/25 kg mortel</td>
</tr>
<tr>
<td>Verbruik per m² circa</td>
<td>1850 kg</td>
</tr>
<tr>
<td>Laagdikte</td>
<td>60 mm max.</td>
</tr>
<tr>
<td>Volumetrische massa</td>
<td>2150 kg/m³</td>
</tr>
<tr>
<td>Spreidmaat</td>
<td>t = 0 min. 140 nm</td>
</tr>
<tr>
<td>Verwerkingstijd</td>
<td>30 minuten</td>
</tr>
<tr>
<td>Zwelling v/v ASTM C627</td>
<td>&gt; 0,1 en &lt; 2,0 %</td>
</tr>
<tr>
<td>Uitdroging krimp NEN 3534 24 uur</td>
<td>&lt; 0,4 mm/m</td>
</tr>
<tr>
<td>Uitdroging krimp NEN 3534 28 dagen</td>
<td>&lt; 1,2 mm/m</td>
</tr>
<tr>
<td>Waterindringing ISO DIS 7031</td>
<td>&lt; 2 mm</td>
</tr>
</tbody>
</table>

Sterkteontwikkeling ISO 679

<table>
<thead>
<tr>
<th>24 uur</th>
<th>7 dagen</th>
<th>28 dagen</th>
<th>91 dagen</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ruigtreksteke N/mm²</td>
<td>5</td>
<td>7</td>
<td>9</td>
</tr>
<tr>
<td>Drukssteke N/mm²</td>
<td>60</td>
<td>75</td>
<td>83</td>
</tr>
<tr>
<td>Drukssteke (150<em>150</em>150 mm) N/mm²</td>
<td></td>
<td></td>
<td>81</td>
</tr>
</tbody>
</table>

Cuglon® bevat geen chloride, noch enige andere corrosie activering stof.

Voor toepassing bij een temperatuur < 0°C dient men contact op te nemen met Cugla B.V.
Appendix B – Kuralon PVA

KURALON™ (PVA fiber)

Better solution for concrete
Characteristics of KURALON™ (PVA fiber)

1. Chemical Structure

```
  CH2    CH    m    CH2    CH    n
  OH     OCOCH3
```

2. Characteristics

High tenacity, High modulus, Low elongation, Light weight, Good resistance against chemicals (alkaline), Good adhesion to cement matrix

<table>
<thead>
<tr>
<th>Properties</th>
<th>Long term durability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fibre</td>
<td>Tensile Strength (MPa)</td>
</tr>
<tr>
<td>----------</td>
<td>------------------------</td>
</tr>
<tr>
<td>KURALON</td>
<td>680 - 1500</td>
</tr>
<tr>
<td>Polypropylene</td>
<td>400</td>
</tr>
<tr>
<td>Steel fibre</td>
<td>1200</td>
</tr>
<tr>
<td>AR-Glass</td>
<td>2200</td>
</tr>
</tbody>
</table>

20 years experience of outdoor weathering test.

Alkaline Resistance

In alkaline environment, calcium hydroxides are easy to adsorb to PVA molecule due to their affinity. (H. Yokoi et al., J. Am. Soc., 106, pp3358-, 1986)
Performance of KURALON™ (1)

3. Improvement for Ductility

Kuralon is difficult to be pulled out from concrete matrix and can bridge between internal surface of cracks. => KURALON™ prevents cracks from growing wider and concrete board can be bent more without breaking.

Concrete

Hard to be bent and easy to be broken

Reinforcement

Metal, Plastics

Hard to be bent and hard to be broken

by KURALON

3-1. Tensile strength and Flexural strength

PVA fibers cannot improve the stress at initial cracking of Tensile and Bending property. However, after initial cracking, they bear the stress instead of concrete matrix. => Kuralon can improve the deformation capacity of concrete board, and especially improve the Bending stress.

Graph 1. Tensile strength

Graph 2. Flexural strength

Table 1. Mix design

<table>
<thead>
<tr>
<th>Fiber dosage</th>
<th>WtBa</th>
<th>Air content</th>
<th>Water</th>
<th>Cement</th>
<th>FA</th>
<th>S</th>
<th>G</th>
<th>Fiber</th>
<th>Ad</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>6</td>
<td>40</td>
<td>3</td>
<td>178</td>
<td>307</td>
<td>131</td>
<td>100</td>
<td>400</td>
<td>5</td>
</tr>
<tr>
<td>1</td>
<td>1117</td>
<td>583</td>
<td>13</td>
<td>480</td>
<td>19.5</td>
<td>25</td>
<td>25</td>
<td>485</td>
<td>32.4</td>
</tr>
<tr>
<td>1.5</td>
<td>1112</td>
<td>482</td>
<td>26</td>
<td>482</td>
<td>25.6</td>
<td>25</td>
<td></td>
<td>485</td>
<td>32.4</td>
</tr>
<tr>
<td>2</td>
<td>1112</td>
<td>482</td>
<td>26</td>
<td>482</td>
<td>25.6</td>
<td>25</td>
<td></td>
<td>485</td>
<td>32.4</td>
</tr>
<tr>
<td>2.5</td>
<td>1112</td>
<td>482</td>
<td>26</td>
<td>482</td>
<td>25.6</td>
<td>25</td>
<td></td>
<td>485</td>
<td>32.4</td>
</tr>
</tbody>
</table>

B = Cement + FA
FA: Fly ash (mineral admixture (addition))
S: Sand (fine aggregate)
G: Gravel (coarse aggregate)
Ad: chemical Admixture
Performance of KURALON™ (2)

3-2. Cracking Control (Multiple micro crack)

In Kuraray's preliminary study, KURALON™ (RSC15x8) 0.6kg/m3 (=1.0 lbs/yd3) can reduce 50% of shrinkage cracking width shown in figure 1. KURALON™ disperses one big crack to many small cracks, and then watertightness and flexural strength of concrete/mortar board can be kept. ECC (Engineered Cementitious Composites) can show the feature in figure 3.

![Graph showing crack opening vs time]

**Fig. 1. Crack opening in 20 °C, 60% R.H. condition**

<table>
<thead>
<tr>
<th>W/C</th>
<th>Sl/a</th>
<th>Water</th>
<th>Cement</th>
<th>Sand</th>
<th>Gravel</th>
<th>SP</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.55</td>
<td>0.50</td>
<td>220</td>
<td>400</td>
<td>807</td>
<td>807</td>
<td>0.30</td>
</tr>
</tbody>
</table>

**Table 1. Mix design: kg/m³**
(Maximum Gravel Size = 10mm)

<table>
<thead>
<tr>
<th></th>
<th>Density (kg/m³)</th>
<th>Slump (cm)</th>
<th>Air Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain Concrete</td>
<td>2.285</td>
<td>19.0</td>
<td>4.8</td>
</tr>
<tr>
<td>RSC15x8 0.6kg/m³ (1.0lbs/yd³)</td>
<td>2.280</td>
<td>18.0</td>
<td>5.0</td>
</tr>
</tbody>
</table>

**Table 2. Fresh properties of concrete**

![Image of ring test specimen]

**Fig. 2. Ring test specimen**

![Image of multiple micro cracks]

**Fig. 3. Multiple micro crack**
Handling of KURALON™

4. How to mix KURALON™ with concrete

※ Example of RF4000x30 with concrete

4-1. Mixing at sight

1. Keeping top speed rotation of the drum of agitating truck.
2. Tossing RF4000x30 in mix, in 10kg/minute tempo.
3. After all of RF4000x30 tossed, add 1 minute rotation of the drum at top speed.

4-2. Mixing at plant

1. Keeping top speed rotation of the plant mixer.
2. Tossing RF4000x30 in mix, in 10kg/minute tempo.
3. After all of RF4000x30 tossed, add 1 minute rotation of the mixer at top speed.

※ Basic concrete must be mixed at first. Never add fibers with Sand, Aggregate, Cement and Water.
Various fineness and length: from 26micron/6mm to 660micron/40mm

- => You can select suitable fibers depending on your mix design

(Normally we recommend the length of fiber is 1.5 times longer than G-max)

Resin-Bundled type: control the dispersibility of fibers to avoid re-aggregating

- During the mixing process (fiber ball)

For Cracking Control: RSC15/8mm

-  

<table>
<thead>
<tr>
<th>Type</th>
<th>Diameter (micron)</th>
<th>Length (mm)</th>
<th>Tensile strength (GPa)</th>
<th>Modulus (GPa)</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>RMS702</td>
<td>26</td>
<td>6</td>
<td>1.6</td>
<td>39</td>
<td>Resin-Bundled type</td>
</tr>
<tr>
<td>RSC15</td>
<td>40</td>
<td>8</td>
<td>1.4</td>
<td>36</td>
<td>Cracking Control</td>
</tr>
<tr>
<td>RECS15</td>
<td>40</td>
<td>8, 12</td>
<td>1.6</td>
<td>41</td>
<td>Resin-Bundled type</td>
</tr>
<tr>
<td>RECS100</td>
<td>100</td>
<td>12</td>
<td>1.2</td>
<td>28</td>
<td>Resin-Bundled type</td>
</tr>
<tr>
<td>RF400</td>
<td>200</td>
<td>6, 12</td>
<td>1.0</td>
<td>27</td>
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</tr>
<tr>
<td>RFS400</td>
<td>200</td>
<td>18</td>
<td>1.0</td>
<td>27</td>
<td>Resin-Bundled type</td>
</tr>
<tr>
<td>RF1000</td>
<td>310</td>
<td>15</td>
<td>1.0</td>
<td>29</td>
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</tr>
<tr>
<td>RF4000</td>
<td>660</td>
<td>30</td>
<td>0.9</td>
<td>23</td>
<td></td>
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</tbody>
</table>

※Keep dry.
※Avoid high temperature and humidity, extremely cold place.
## Applications of KURALON™

<table>
<thead>
<tr>
<th>Application</th>
<th>Fiber Type</th>
<th>Fiber Content</th>
<th>Functions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building Wall (Cladding Wall)</td>
<td>RMS702, RECS15</td>
<td>1 - 1.5 vol%</td>
<td>Toughness, Cracking control</td>
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<tr>
<td>Permanent Formworks (Precast)</td>
<td>RECS15, RF4000</td>
<td>2 - 2.5 vol%</td>
<td>Toughness, Cracking control</td>
</tr>
<tr>
<td>Bridge Deck Slab</td>
<td>RECS100</td>
<td>0.075 vol%</td>
<td>Cracking control, Abrasion prevention</td>
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<tr>
<td>OA Panel for Floor</td>
<td>RECS100, RF400</td>
<td>2.5 - 3 vol%</td>
<td>Impact strength, Toughness</td>
</tr>
<tr>
<td>Wind Proof Panel for Railways</td>
<td>RFS400</td>
<td>2.5 vol%</td>
<td>Cracking control, Toughness</td>
</tr>
<tr>
<td>Retrofit</td>
<td>RMS702, RECS15, RECS100</td>
<td>0.1 - 2 vol%</td>
<td>Cracking control, Toughness</td>
</tr>
<tr>
<td>Shotcrete for Slope Stabilization</td>
<td>RFS400, RF4000</td>
<td>0.75 - 1 vol%</td>
<td>Toughness, Cracking control</td>
</tr>
<tr>
<td>Shotcrete for Tunnel Lining</td>
<td>RF4000</td>
<td>0.75 - 1 vol%</td>
<td>Toughness</td>
</tr>
<tr>
<td>Overlay for Concrete Road Pavement</td>
<td>RF4000</td>
<td>0.75 vol%</td>
<td>Toughness, Frost damage resistance</td>
</tr>
<tr>
<td>Concrete Slab on Grade</td>
<td>RECS100, RF4000</td>
<td>0.15, 0.46 vol%</td>
<td>Cracking control, Toughness</td>
</tr>
<tr>
<td>Heavy Concrete Floor Surface</td>
<td>RSC15</td>
<td>0.077 vol%</td>
<td>Cracking control</td>
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</tbody>
</table>

**Curtain Wall**

**Permanent Formwork**

**OA Floor Panel**

**Shotcretin for Slope Stabilization**

**Tunnel Lining**
URL: http://www.kuraray.co.jp/en

Fibers and Industrial Materials Division
Ote Center Building, 1-1-3, Otemachi, Chiyoda-ku,
Tokyo 100-8115, Japan
TEL; +81-3-6701-1372
FAX; +81-3-6701-1376
## Appendix C – Results of Preliminary Tests

### Compression Strength

#### Table C1: Compression strength of mixtures in preliminary test

<table>
<thead>
<tr>
<th>t[days]</th>
<th>Mixture</th>
<th>C</th>
<th>S1</th>
<th>S2</th>
<th>S1P1</th>
<th>S1P2</th>
<th>S1P3</th>
<th>S2P1</th>
<th>S2P2</th>
<th>S2P3</th>
<th>P1</th>
<th>P2</th>
<th>P3</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>Prism I</td>
<td>65.50</td>
<td>67.80</td>
<td>72.80</td>
<td>72.20</td>
<td>67.70</td>
<td>70.90</td>
<td>70.70</td>
<td>72.20</td>
<td>71.80</td>
<td>65.30</td>
<td>64.30</td>
<td>69.60</td>
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<tr>
<td>7</td>
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<td>65.20</td>
<td>63.20</td>
<td>74.10</td>
<td>68.80</td>
<td>66.30</td>
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<td>73.20</td>
<td>73.90</td>
<td>65.60</td>
<td>68.40</td>
<td>65.80</td>
</tr>
<tr>
<td>7</td>
<td>Prism II</td>
<td>67.20</td>
<td>66.40</td>
<td>72.20</td>
<td>70.40</td>
<td>68.60</td>
<td>70.20</td>
<td>72.50</td>
<td>74.30</td>
<td>69.10</td>
<td>65.60</td>
<td>66.00</td>
<td>62.90</td>
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<td>64.80</td>
<td>71.00</td>
<td>65.60</td>
<td>66.00</td>
<td>62.90</td>
<td>65.60</td>
<td>66.00</td>
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<td>71.45</td>
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<td>-1%</td>
<td>-6%</td>
<td>-4%</td>
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<tr>
<td></td>
<td>[A] Average 7 days</td>
<td>66.00</td>
<td>65.60</td>
<td>72.33</td>
<td>70.18</td>
<td>67.28</td>
<td>71.13</td>
<td>72.48</td>
<td>72.82</td>
<td>71.45</td>
<td>65.28</td>
<td>64.83</td>
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#### Percentage Increase

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<table>
<thead>
<tr>
<th>t[days]</th>
<th>Mixture</th>
<th>C</th>
<th>S1</th>
<th>S2</th>
<th>S1P1</th>
<th>S1P2</th>
<th>S1P3</th>
<th>S2P1</th>
<th>S2P2</th>
<th>S2P3</th>
<th>P1</th>
<th>P2</th>
<th>P3</th>
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<tbody>
<tr>
<td>28</td>
<td>Prism I</td>
<td>81.80</td>
<td>80.50</td>
<td>83.40</td>
<td>78.40</td>
<td>86.70</td>
<td>83.00</td>
<td>87.90</td>
<td>88.50</td>
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<td>76.50</td>
<td>76.40</td>
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<td>28</td>
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<td>86.90</td>
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<td>85.90</td>
<td>85.00</td>
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<td>81.90</td>
<td>81.90</td>
<td>89.40</td>
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</tr>
<tr>
<td>28</td>
<td>Prism III</td>
<td>81.20</td>
<td>82.40</td>
<td>86.70</td>
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<td>82.40</td>
<td>86.30</td>
<td>86.00</td>
<td>80.80</td>
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<td>80.30</td>
<td>82.10</td>
<td>87.00</td>
<td>71.10</td>
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<td></td>
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<td></td>
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<tr>
<td>[B] Average 28 days</td>
<td>80.17</td>
<td>81.45</td>
<td>86.70</td>
<td>80.00</td>
<td>82.73</td>
<td>82.88</td>
<td>87.18</td>
<td>87.02</td>
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#### Percentage Increase

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<tr>
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<tr>
<td>-4%</td>
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<td>-3%</td>
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#### Percentage Increase wrt 7 days

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</tr>
<tr>
<td>15%</td>
</tr>
<tr>
<td>19%</td>
</tr>
<tr>
<td>19%</td>
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</table>
Figure 157: Average compression strength values with standard deviation
## Flexural Strength

### Table C2: Flexural strength of mixtures in preliminary test

<table>
<thead>
<tr>
<th>t[days]</th>
<th>Flexure [N/mm²]</th>
<th>C</th>
<th>S1</th>
<th>S2</th>
<th>S1P1</th>
<th>S1P2</th>
<th>S1P3</th>
<th>S2P1</th>
<th>S2P2</th>
<th>S2P3</th>
<th>P1</th>
<th>P2</th>
<th>P3</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>Prism I</td>
<td>10.72</td>
<td>12.98</td>
<td>13.86</td>
<td>12.38</td>
<td>11.02</td>
<td>11.65</td>
<td>11.87</td>
<td>11.67</td>
<td>11.52</td>
<td>10.98</td>
<td>9.93</td>
<td>7.7</td>
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<tr>
<td>7</td>
<td>Prism III</td>
<td>9.33</td>
<td>12</td>
<td>12</td>
<td>12.07</td>
<td>10.09</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>Average 7 days</td>
<td>10.08</td>
<td>12.87</td>
<td>12.39</td>
<td>12.31</td>
<td>11.53</td>
<td>11.89</td>
<td>12.67</td>
<td>12.56</td>
<td>11.51</td>
<td>11.58</td>
<td>10.21</td>
<td>8.41</td>
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<td>14%</td>
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<td>26%</td>
<td>25%</td>
<td>14%</td>
<td>15%</td>
<td>1%</td>
<td>-17%</td>
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<td>Prism III</td>
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<td>13.59</td>
<td>12.43</td>
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<td></td>
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<td></td>
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<tr>
<td>B</td>
<td>Average 28 days</td>
<td>12.04</td>
<td>13.05</td>
<td>13.82</td>
<td>13.27</td>
<td>12.88</td>
<td>13.20</td>
<td>13.60</td>
<td>13.39</td>
<td>14.04</td>
<td>12.71</td>
<td>13.02</td>
<td>12.41</td>
</tr>
<tr>
<td></td>
<td>Percentage Increase</td>
<td>0%</td>
<td>8%</td>
<td>15%</td>
<td>10%</td>
<td>7%</td>
<td>10%</td>
<td>13%</td>
<td>11%</td>
<td>17%</td>
<td>5%</td>
<td>8%</td>
<td>3%</td>
</tr>
<tr>
<td></td>
<td>Percentage Increase to 7 days</td>
<td>19%</td>
<td>1%</td>
<td>12%</td>
<td>8%</td>
<td>12%</td>
<td>11%</td>
<td>7%</td>
<td>7%</td>
<td>22%</td>
<td>10%</td>
<td>28%</td>
<td>48%</td>
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</table>
Average Flexural Strength

Figure 158: Average flexural strength values with standard deviation
Appendix D – Results of Secondary Tests

Compression, Splitting Tensile and Flexural Strength

| Table D1: Compression and Splitting Tensile Strength of Mixtures in Secondary Test |
|-----------------|----------------|--------|--------|--------|--------|--------|
| t[days] Mixture | S1  | S2  | S1P1  | S2P1  |        |        |
| Compression [N/mm²] 8 Cube I | 62.72 | 62.67 | 62.97 | 59.53 |        |        |
| 8 Cube II | 58.98 | 65.28 | 58.80 | 64.19 |        |        |
| 8 Cube III | 57.22 | 59.16 | 63.96 | 64.58 |        |        |
| [A] Average | **59.64** | **62.37** | **61.91** | **62.77** |        |        |
| Percentage Increase | 0% | 5% | 4% | 5% |        |        |
| Splitting Tensile [N/mm²] 8 Cube I | 4.99 | 5.31 | 4.92 | 5.93 |        |        |
| 8 Cube II | 5.06 | 5.55 | 4.99 | - |        |        |
| 8 Cube III | 4.58 | - | 5.12 | 5.76 |        |        |
| [B] Average | **4.88** | **5.43** | **5.01** | **5.85** |        |        |
| Percentage Increase | 0% | 11% | 3% | 20% |        |        |
| Flexural [N/mm²] 8 Prism I | - | - | - | - | 6.02 | 5.99 |
| 8 Prism II | 4.81 | 5.83 | 6.06 | 6.38 |        |        |
| 8 Prism III | 5.43 | 6.06 | 5.94 |        |        |        |
| [C] Average | **5.12** | **5.83** | **6.04** | **6.10** |        |        |
| Percentage Increase | 0% | 14% | 18% | 19% |        |        |
Flexural and Residual Flexural Behaviour of Mixture S1

<table>
<thead>
<tr>
<th>CMOD [mm]</th>
<th>δ [mm]</th>
<th>Ft [kN]</th>
<th>f [N/mm²]</th>
<th>S [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-02</td>
<td>0.079</td>
<td>0.067</td>
<td>15.03</td>
<td>4.81</td>
</tr>
<tr>
<td>S1-03</td>
<td>0.033</td>
<td>0.065</td>
<td>16.97</td>
<td>5.43</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td>16.00</td>
<td>5.12</td>
</tr>
</tbody>
</table>

CMOD1= 0,5 mm  
CMOD2= 1 mm  
CMOD3= 1,5 mm  
CMOD4= 2 mm

<table>
<thead>
<tr>
<th>CMOD1= 0,5 mm</th>
<th>CMOD2= 1 mm</th>
<th>CMOD3= 1,5 mm</th>
<th>CMOD4= 2 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-02 F [kN]</td>
<td>10.9</td>
<td>7.51</td>
<td>6.87</td>
</tr>
<tr>
<td>f [N/mm²]</td>
<td>3.49</td>
<td>2.40</td>
<td>2.20</td>
</tr>
<tr>
<td>S1-03 F [kN]</td>
<td>8.42</td>
<td>8.08</td>
<td>7.46</td>
</tr>
<tr>
<td>f [N/mm²]</td>
<td>2.69</td>
<td>2.59</td>
<td>2.39</td>
</tr>
</tbody>
</table>

Figure 159: Vertical Displacement and CMOD of mixture S1
Flexural and Residual Flexural Behaviour of Mixture S2

Table D3: (Residual) Flexural strength of specimens of mixture S2

<table>
<thead>
<tr>
<th>CMOD [mm]</th>
<th>δ [mm]</th>
<th>F [kN]</th>
<th>f [N/mm^2]</th>
<th>S [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2-02</td>
<td>0.02</td>
<td>0.06</td>
<td>18.22</td>
<td>5.83</td>
</tr>
</tbody>
</table>

CMOD1= 0,5 mm  CMOD2= 1 mm  CMOD3= 1,5 mm  CMOD4= 2 mm

| S2-02 | F [kN] | 14.30 | 14.47 | 13.54 | 12.65 |
|       | f [N/mm2] | 4.57 | 4.63 | 4.33 | 4.05 |

Figure 160: Vertical Displacement and CMOD of mixture S2
Figure 161: Incomplete graphs of specimen S2-03; graphs display the residual strength of the specimen.
## Flexural and Residual Flexural Behaviour of Mixture S1P1

### Table D4: (Residual) Flexural strength of specimens of mixture S1P1

<table>
<thead>
<tr>
<th>CMOD [mm]</th>
<th>δ [mm]</th>
<th>F [kN]</th>
<th>f [N/mm(^2)]</th>
<th>S [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1P1-01</td>
<td>0.02</td>
<td>0.06</td>
<td>18.80</td>
<td>6.02</td>
</tr>
<tr>
<td>S1P1-02</td>
<td>0.02</td>
<td>0.07</td>
<td>18.92</td>
<td>6.06</td>
</tr>
<tr>
<td>S1P1-03</td>
<td>0.02</td>
<td>0.06</td>
<td>18.92</td>
<td>6.06</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td></td>
<td>18.88</td>
<td>6.04</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>CMOD = 0.5 mm</th>
<th>CMOD = 1 mm</th>
<th>CMOD = 1.5 mm</th>
<th>CMOD = 2 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>F [kN]</td>
<td>F [kN]</td>
<td>F [kN]</td>
<td>F [kN]</td>
</tr>
<tr>
<td>f [N/mm(^2)]</td>
<td>f [N/mm(^2)]</td>
<td>f [N/mm(^2)]</td>
<td>f [N/mm(^2)]</td>
</tr>
<tr>
<td>S1P1-01</td>
<td>7.56</td>
<td>7.75</td>
<td>7.58</td>
</tr>
<tr>
<td>S1P1-02</td>
<td>9.02</td>
<td>9.34</td>
<td>8.97</td>
</tr>
<tr>
<td>S1P1-03</td>
<td>2.89</td>
<td>2.99</td>
<td>2.87</td>
</tr>
<tr>
<td>Average Force [kN]</td>
<td>8.62</td>
<td>8.79</td>
<td>8.50</td>
</tr>
<tr>
<td>Average Stress [N/mm(^2)]</td>
<td>2.76</td>
<td>2.81</td>
<td>2.72</td>
</tr>
</tbody>
</table>
Figure 162: Vertical Displacement and CMOD of mixture S1P1; specimen I

Figure 163: Vertical Displacement and CMOD of mixture S1P1; specimen II
Figure 164: Vertical Displacement and CMOD of mixture S1P1; specimen III
### Flexural and Residual Flexural Behaviour of Mixture S2P1

#### Table D5: (Residual) Flexural Strength of specimens of mixture S2P1

<table>
<thead>
<tr>
<th></th>
<th>CMOD [mm]</th>
<th>δ [mm]</th>
<th>Ft [kN]</th>
<th>f [N/mm²]</th>
<th>S [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2P1-01</td>
<td>0.022</td>
<td>0.062</td>
<td>18.71</td>
<td>5.99</td>
<td>4.35</td>
</tr>
<tr>
<td>S2P1-02</td>
<td>0.023</td>
<td>0.066</td>
<td>19.92</td>
<td>6.38</td>
<td>4.52</td>
</tr>
<tr>
<td>S2P1-03</td>
<td>0.024</td>
<td>0.066</td>
<td>18.57</td>
<td>5.94</td>
<td>4.17</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td></td>
<td>19.07</td>
<td>6.10</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>CMOD1= 0.5 mm</th>
<th>CMOD2= 1 mm</th>
<th>CMOD3= 1.5 mm</th>
<th>CMOD4= 2 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2P1-01</td>
<td>10.96</td>
<td>10.92</td>
<td>10.45</td>
<td>9.60</td>
</tr>
<tr>
<td>f [N/mm²]</td>
<td>3.51</td>
<td>3.49</td>
<td>3.34</td>
<td>3.07</td>
</tr>
<tr>
<td>S2P1-02</td>
<td>13.58</td>
<td>13.87</td>
<td>12.90</td>
<td>11.76</td>
</tr>
<tr>
<td>f [N/mm²]</td>
<td>4.34</td>
<td>4.44</td>
<td>4.13</td>
<td>3.76</td>
</tr>
<tr>
<td>S2P1-03</td>
<td>16.19</td>
<td>16.78</td>
<td>15.71</td>
<td>13.97</td>
</tr>
<tr>
<td>f [N/mm²]</td>
<td>5.18</td>
<td>5.37</td>
<td>5.03</td>
<td>4.47</td>
</tr>
<tr>
<td><strong>Average Force [kN]</strong></td>
<td>13.57</td>
<td>13.86</td>
<td>13.02</td>
<td>11.78</td>
</tr>
<tr>
<td><strong>Average Stress [N/mm²]</strong></td>
<td>4.34</td>
<td>4.43</td>
<td>4.17</td>
<td>3.77</td>
</tr>
</tbody>
</table>
Figure 165: Vertical Displacement and CMOD of mixture S2P1; specimen I

Figure 166: Vertical Displacement and CMOD of mixture S2P1; specimen II
Figure 167: Vertical Displacement and CMOD of mixture S2P1; specimen III
Comparison of Flexural Behaviour of Mixtures

**Vertical Displacement mixtures S1 and S2**

![Graph showing vertical displacement](image1)

*Figure 168: Vertical Displacement of mixtures S1 and S2*

**CMOD mixtures S1 and S2**

![Graph showing CMOD](image2)

*Figure 169: CMOD of mixtures S1 and S2*
Figure 170: Vertical Displacement of mixtures S1P1 and S2P1

Figure 171: CMOD of mixtures S1P1 and S2P1
Figure 172: Vertical Displacement of mixtures S1 and S1P1

Figure 173: CMOD of mixtures S1 and S1P1
Figure 174: Vertical Displacement of mixtures S2 and S2P1

Figure 175: CMOD of mixtures S2 and S2P1
Appendix E – Results of Tertiary Tests

Table E1: Results of the compression tests for the pumped mixtures at 7 days

<table>
<thead>
<tr>
<th>mix</th>
<th>Prism 1</th>
<th>Prism 2</th>
<th>Prism 3</th>
<th>Prism 4</th>
<th>Prism 5</th>
<th>Prism 6</th>
<th>Prism 7</th>
<th>Prism 8</th>
<th>Prism 9</th>
<th>Prism 10</th>
<th>Prism 11</th>
<th>Prism 12</th>
<th>Average [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>66.4</td>
<td>66.6</td>
<td>71.5</td>
<td>70.7</td>
<td>70.3</td>
<td>72.7</td>
<td>70.0</td>
<td>70.8</td>
<td>67.8</td>
<td>66.4</td>
<td>70.1</td>
<td>72.4</td>
<td>63.6</td>
</tr>
<tr>
<td>S1</td>
<td>73.6</td>
<td>79.2</td>
<td>72.1</td>
<td>74.0</td>
<td>74.7</td>
<td>72.5</td>
<td>71.3</td>
<td>71.7</td>
<td>75.0</td>
<td>69.0</td>
<td>76.2</td>
<td>75.2</td>
<td>74.3</td>
</tr>
<tr>
<td>S1.5</td>
<td>77.4</td>
<td>73.1</td>
<td>74.7</td>
<td>74.8</td>
<td>76.0</td>
<td>74.3</td>
<td>77.0</td>
<td>73.6</td>
<td>73.1</td>
<td>73.1</td>
<td>76.1</td>
<td>74.5</td>
<td>77.6</td>
</tr>
<tr>
<td>S2</td>
<td>77.2</td>
<td>78.5</td>
<td>77.3</td>
<td>67.3</td>
<td>75.0</td>
<td>75.9</td>
<td>78.4</td>
<td>74.4</td>
<td>75.6</td>
<td>77.9</td>
<td>76.7</td>
<td>73.3</td>
<td>77.5</td>
</tr>
<tr>
<td>S2P1</td>
<td>75.5</td>
<td>78.7</td>
<td>74.4</td>
<td>77.5</td>
<td>83.9</td>
<td>80.6</td>
<td>77.5</td>
<td>80.7</td>
<td>76.5</td>
<td>77.7</td>
<td>82.8</td>
<td>80.7</td>
<td>77.9</td>
</tr>
</tbody>
</table>

Table E2: Results of the flexural tests for the pumped mixtures at 7 days

<table>
<thead>
<tr>
<th>mix</th>
<th>Prism 1</th>
<th>Prism 2</th>
<th>Prism 3</th>
<th>Prism 4</th>
<th>Prism 5</th>
<th>Prism 6</th>
<th>Prism 7</th>
<th>Prism 8</th>
<th>Prism 9</th>
<th>Prism 10</th>
<th>Prism 11</th>
<th>Prism 12</th>
<th>Average [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1.5</td>
<td>10.19</td>
<td>9.37</td>
<td>10.15</td>
<td>9.00</td>
<td>11.25</td>
<td>13.08</td>
<td>14.08</td>
<td>11.28</td>
<td>12.75</td>
<td>11.74</td>
<td>14.42</td>
<td>7.27</td>
<td>11.20</td>
</tr>
</tbody>
</table>
Appendix F – Set-Up Testing Machine
Appendix G – Photos of Specimens and Test Set-Up

Figure 176: 3 in 1 steel moulds for used in the first and third phase

Figure 177: Specimens before de-moulding

Figure 178: Notching of the beams by means of a diamond saw
Figure 179: Specimens recently taken from the fog room
Figure 180: Overview of test set-up

Figure 181: the cracked surface and protruding fibres
Figure 182: Compressed cubes (left and upper right) and split cubes (lower right)
Appendix H – FEA Input Parameters for SFRM

Table H1: Input parameters 3DNonLinCementitious2User (3D NLC2U)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive properties (compressive softening)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>fc reduction due to cracks</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ε</td>
<td>σc/fc</td>
<td></td>
<td>Function defining the compressive strength reduction due to cracking. Strains represent fracturing strains normal to a crack</td>
</tr>
<tr>
<td>0</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00E-03</td>
<td>1</td>
<td></td>
<td>These are default values generated by ATENA</td>
</tr>
<tr>
<td>5.00E-03</td>
<td>0.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00E-02</td>
<td>0.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.50E-02</td>
<td>0.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.00E-02</td>
<td>0.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.00E-01</td>
<td>0.02</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear properties</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ε</td>
<td>G/Gc</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00E-05</td>
<td>0.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.00E-04</td>
<td>0.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00E-03</td>
<td>0.15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.00E-03</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ε_p</td>
<td></td>
<td>0</td>
<td>Strain at localization; default value is equal to 0</td>
</tr>
<tr>
<td>τ reduction due to cracks</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ε</td>
<td>τ/t</td>
<td></td>
<td>Function defining the shear strength of cracked concrete based on crack width in the crack direction. Strains represent fracturing strains normal to a crack</td>
</tr>
<tr>
<td>0</td>
<td>1.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00E-03</td>
<td>0.87</td>
<td></td>
<td>These are default values generated by ATENA</td>
</tr>
<tr>
<td>5.00E-03</td>
<td>0.51</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00E-02</td>
<td>0.34</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.00E-03</td>
<td>0.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.00E-03</td>
<td>0.15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.00E-03</td>
<td>0.09</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00E-02</td>
<td>0.05</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension-compression interaction</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>α_c/σ_t</td>
<td></td>
<td></td>
<td>Function defining the tensile strength reduction based on the compressive stress in other material directions.</td>
</tr>
<tr>
<td>0</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Miscellaneous</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Exc</td>
<td></td>
<td>0.52</td>
<td>Excentricity, defining the shape of the failure surface; default 0.52</td>
</tr>
<tr>
<td>β</td>
<td></td>
<td>0</td>
<td>Multiplier for the direction of the plastic flow; default 0</td>
</tr>
<tr>
<td>ρ</td>
<td>mN/m³</td>
<td>2.15E-02</td>
<td>Specific material density of mortar</td>
</tr>
<tr>
<td>α</td>
<td></td>
<td>1.20E-05</td>
<td>Coefficient of thermal expansion</td>
</tr>
<tr>
<td>Fix.cr mod</td>
<td></td>
<td>0.25</td>
<td>Factor for the fixed smeared crack model, default value 0.25</td>
</tr>
</tbody>
</table>
Figure 183: Typical dialog box for input of tension properties and tension softening (bilinear) function

Figure 184: Typical dialog box for input of compression properties and compression softening (bilinear) function
Macro-elements Input Parameters

Macro-elements are displayed for the beam. The mesh type, element sizes, material and thickness of the macro-elements are displayed in Table 1.

<table>
<thead>
<tr>
<th>Macro-el</th>
<th>Mesh type</th>
<th>Elements</th>
<th>Element size [m]</th>
<th>Material</th>
<th>Thickness [m]</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Quadrilateral</td>
<td>CCIsoQuad</td>
<td>0.01</td>
<td>Steel</td>
<td>0.15</td>
<td>Steel plate</td>
</tr>
<tr>
<td>2</td>
<td>Quadrilateral</td>
<td>CCIsoQuad</td>
<td>0.015</td>
<td>3D NLC2U</td>
<td>0.15</td>
<td>SFRM</td>
</tr>
<tr>
<td>3</td>
<td>Quadrilateral</td>
<td>CCIsoQuad</td>
<td>0.01</td>
<td>Steel</td>
<td>0.15</td>
<td>Steel plate</td>
</tr>
<tr>
<td>4</td>
<td>Quadrilateral</td>
<td>CCIsoQuad</td>
<td>0.015</td>
<td>3D NLC2U</td>
<td>0.15</td>
<td>SFRM</td>
</tr>
<tr>
<td>5</td>
<td>Quadrilateral</td>
<td>CCIsoQuad</td>
<td>0.01</td>
<td>3D NLC2U</td>
<td>0.15</td>
<td>SFRM</td>
</tr>
</tbody>
</table>

Load Cases Input Parameters

1. Load case with support
2. Load case with actions
   a. Prescribed deformation; y = -2E-05 m
   b. constant

Solution Parameters

Newton-Raphson method with line search: NR w/LS

a. General:
   i. Optimize node numbers: Sloan
   ii. Update stiffness: each iteration
   iii. Stiffness type: tangent
   iv. Iteration number limit: 50
   v. Displacement error tolerance: 0.01
   vi. Residual error tolerance: 0.01
   vii. Absolute residual error tolerance: 0.01
   viii. Energy error tolerance: 0.0001

b. Line search: with iterations
   i. Unbalanced energy limit: 0.8
   ii. Limit of line search iterations: 20
   iii. Line search limit - min: 0.1
   iv. Line search limit - max: 10
## Appendix I – FEA Input Parameters for Profiled Mortar Joint

### Table I: Input parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcement</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>MPa</td>
<td>200000</td>
<td>Elastic Modulus</td>
</tr>
<tr>
<td>σ_y</td>
<td>MPa</td>
<td>500.00</td>
<td>Yield strength</td>
</tr>
<tr>
<td>ρ</td>
<td>MN/m³</td>
<td>7.85E-02</td>
<td>Specific material weight</td>
</tr>
<tr>
<td>α</td>
<td>1/K</td>
<td>1.20E-05</td>
<td>Coefficient of thermal expansion</td>
</tr>
<tr>
<td>Bond for Reinforcement</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slip [m]</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td></td>
<td>7.58E+00</td>
<td>Fracture strain at tensile strength</td>
</tr>
<tr>
<td>2</td>
<td>2.50E-04</td>
<td>9.95E+00</td>
<td>Ratio stress to tensile strength at corresponding fracture strain</td>
</tr>
<tr>
<td>3</td>
<td>5.00E-04</td>
<td>1.44E+01</td>
<td>Fracture strain corresponding to ( \varepsilon_c ); ( \varepsilon_2 = w_c/\bar{l}_i, \bar{l}<em>i=l</em>{ch} )</td>
</tr>
<tr>
<td>4</td>
<td>1.00E-03</td>
<td>1.89E-01</td>
<td>Fracture strain to tensile strength; ( f_{t,eq,bil}/f_{t,ax} )</td>
</tr>
<tr>
<td>5</td>
<td>3.00E-03</td>
<td>1.89E-01</td>
<td>Fracture strain corresponding to ( \varepsilon_3 ); ( \varepsilon_3 = w_0/\bar{l}_i, \bar{l}<em>i=l</em>{ch} )</td>
</tr>
<tr>
<td>6</td>
<td>1.50E-02</td>
<td>7.58E+00</td>
<td>Ratio stress to tensile strength; ( f_{t,eq,bil}/f_{t,ax} )</td>
</tr>
<tr>
<td>7</td>
<td>1.00E+00</td>
<td>7.58E+00</td>
<td>Characteristic size is assumed equal to influence length; ( l_{ch}=l_i )</td>
</tr>
</tbody>
</table>

| 2D Interface |        |           |                                                                             |
| Knn         | MN/m³  | 2.00E+08 | Normal Stiffness                                                           |
| Ktt         | MN/m³  | 2.00E+08 | Tangential stiffness                                                        |
| ft          | MPa    | 0.00E+00 | Tensile Strength                                                           |
| Cohesion    | MPa    | 0        |                                                                             |
| φ           |        | 0.00E+00 | Friction coefficient                                                        |

| Steel Supports |        |           |                                                                             |
| E             | MPa    | 2.10E+05 | Elastic Modulus                                                            |
| μ             | -      | 0.30     | Poisson’s ratio                                                             |
| ρ             | MN/m³  | 7.85E-02 | Specific material weight                                                    |
| α             | 1/K    | 1.20E-05 | Coefficient of thermal expansion                                            |

| Pre-stressing cable |        |           |                                                                             |
| ρ             | MN/m³  | 7.85E-02 | Specific material weight                                                    |
| α             | 1/K    | 1.20E-05 | Coefficient of thermal expansion                                            |

| Concrete Elements |        |           |                                                                             |
| E             | MPa    | 4.03E+04 |                                                                             |
| μ             | -      | 0.20     | Poisson’s ratio                                                             |
| ft            | MPa    | 3.88     | Tensile Strength                                                            |
| fc            | MPa    | -5.53E+01| Cylinder Compressive Strength                                               |

| Type of tension softening |        |           |                                                                             |
| Gf            | MN/m   | 9.70E-05 | Specific fracture energy                                                    |

| Compressive |        |           |                                                                             |
| ε_c          | -      | -2.75E-03| Compressive strain at compressive strength                                  |
| a            | -      | 0.8      | Reduction of compressive strength due to cracks                             |

| Type of compression softening |        |           |                                                                             |
| w₀           | m      | -5.00E-04| Critical compressive displacement                                           |

| Shear |        |           |                                                                             |
| Variable |        |           |                                                                             |

| Tension-compression interaction |        |           |                                                                             |
| ρ             | MN/m³  | 2.30E-02 | Specific material weight                                                    |
| α             | 1/K    | 1.20E-05 | Coefficient of thermal expansion                                            |
Table I2: Input data for macro-elements 3D NLC2U=3DNonLinCementitious2User

<table>
<thead>
<tr>
<th>Description</th>
<th>Mesh type</th>
<th>Elements</th>
<th>Element size [m]</th>
<th>Material</th>
<th>Thickness [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel support</td>
<td>Quadrilateral</td>
<td>CCIsoQuad</td>
<td>0.03</td>
<td>Steel</td>
<td>0.2</td>
</tr>
<tr>
<td>Mortar</td>
<td>Quadrilateral</td>
<td>CCIsoQuad</td>
<td>0.01</td>
<td>3D NLC2U</td>
<td>0.2</td>
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<tr>
<td>Concrete</td>
<td>Quadrilateral</td>
<td>CCIsoQuad</td>
<td>0.05</td>
<td>SBETA</td>
<td>0.2</td>
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<tr>
<td>Concrete Fracture zone</td>
<td>Quadrilateral</td>
<td>CCIsoQuad</td>
<td>0.0125</td>
<td>SBETA</td>
<td>0.2</td>
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Load Cases Input Parameters
3. Load case with support
4. Load case with actions
   a. Prescribed deformation; y = -1E-04 m
   b. constant
5. Pre-Stressing: 6E-03 mN per steel rod

Solution Parameters
Newton-Raphson method with line search: NR w/LS
   c. General:
      i. Optimize node numbers: Sloan
      ii. Update stiffness: each iteration
      iii. Stiffness type: tangent
      iv. Iteration number limit: 40
      v. Displacement error tolerance: 0.01
      vi. Residual error tolerance: 0.01
      vii. Absolute residual error tolerance: 0.01
      viii. Energy error tolerance: 0.0001
   d. Line search: with iterations
      i. Unbalanced energy limit: 0.8
      ii. Limit of line search iterations: 20
      iii. Line search limit - min: 0.1
      iv. Line search limit - max: 10
## Glossary of Terms

### Abbreviations

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
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<tbody>
<tr>
<td>ACI</td>
<td>American Concrete Institute</td>
</tr>
<tr>
<td>ASCE</td>
<td>American Society of Civil engineers</td>
</tr>
<tr>
<td>BSI</td>
<td>British Standard Institute</td>
</tr>
<tr>
<td>CMOD</td>
<td>Crack Mouth Opening Displacement</td>
</tr>
<tr>
<td>CoV</td>
<td>Coefficient of Variation</td>
</tr>
<tr>
<td>EC II</td>
<td>NEN-EN 1992-1-1 Eurocode II: Design of concrete structures</td>
</tr>
<tr>
<td>FCM</td>
<td>Fictitious Crack Model</td>
</tr>
<tr>
<td>FEA</td>
<td>Finite Element Analysis</td>
</tr>
<tr>
<td>FEM</td>
<td>Finite Element Model</td>
</tr>
<tr>
<td>FR</td>
<td>Fibre Reinforced</td>
</tr>
<tr>
<td>FRC</td>
<td>Fibre Reinforced Concrete</td>
</tr>
<tr>
<td>FRCC</td>
<td>Fibre Reinforced Cementitious Composite</td>
</tr>
<tr>
<td>FRSCC</td>
<td>Fibre Reinforced Self Compacting Concrete</td>
</tr>
<tr>
<td>PVA</td>
<td>Poly Vinyl Alcohol</td>
</tr>
<tr>
<td>SFRC</td>
<td>Steel Fibre Reinforced Concrete</td>
</tr>
<tr>
<td>SFRM</td>
<td>Steel Fibre Reinforced Mortar</td>
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### Latin Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
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<tbody>
<tr>
<td>a</td>
<td>Notch height</td>
</tr>
<tr>
<td>A&lt;sub&gt;eff&lt;/sub&gt;</td>
<td>Effective area</td>
</tr>
<tr>
<td>b</td>
<td>Beam width</td>
</tr>
<tr>
<td>D</td>
<td>Toughness</td>
</tr>
<tr>
<td>E&lt;sub&gt;sp&lt;/sub&gt;</td>
<td>Secant modulus at peak stress</td>
</tr>
<tr>
<td>E&lt;sub&gt;t&lt;/sub&gt;</td>
<td>Tangent modulus</td>
</tr>
<tr>
<td>f&lt;sub&gt;ccm&lt;/sub&gt;</td>
<td>Linear elastic compression strength</td>
</tr>
<tr>
<td>f&lt;sub&gt;ctm,ax&lt;/sub&gt;</td>
<td>Uniaxial tensile strength</td>
</tr>
<tr>
<td>f&lt;sub&gt;ctm,eq,bil&lt;/sub&gt;</td>
<td>Bilinear equivalent post-cracking strength</td>
</tr>
<tr>
<td>f&lt;sub&gt;ctm,spil&lt;/sub&gt;</td>
<td>Splitting tensile strength</td>
</tr>
<tr>
<td>f&lt;sub&gt;mk(7)&lt;/sub&gt;</td>
<td>mortar compression strength at 7 days</td>
</tr>
<tr>
<td>FRI</td>
<td>Fibre reinforcement Index</td>
</tr>
<tr>
<td>FRI&lt;sub&gt;2&lt;/sub&gt;</td>
<td>Extended fibre reinforcement index</td>
</tr>
<tr>
<td>G&lt;sub&gt;fc&lt;/sub&gt;</td>
<td>Energy absorption capacity</td>
</tr>
<tr>
<td>h</td>
<td>Beam depth</td>
</tr>
<tr>
<td>h&lt;sub&gt;lig&lt;/sub&gt;</td>
<td>Effective beam depth</td>
</tr>
<tr>
<td>L&lt;sub&gt;d&lt;/sub&gt;</td>
<td>Span length</td>
</tr>
<tr>
<td>L&lt;sub&gt;e&lt;/sub&gt;</td>
<td>Embedded fibre length</td>
</tr>
<tr>
<td>L&lt;sub&gt;f&lt;/sub&gt;</td>
<td>Fibre length</td>
</tr>
<tr>
<td>l&lt;sub&gt;i&lt;/sub&gt;</td>
<td>Influence length</td>
</tr>
<tr>
<td>P&lt;sub&gt;c(6)&lt;/sub&gt;</td>
<td>Computed load</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>----------</td>
<td>--------------------------------------------------</td>
</tr>
<tr>
<td>$P_M(\delta)$</td>
<td>Measured load</td>
</tr>
<tr>
<td>$w_0$</td>
<td>Critical crack width</td>
</tr>
<tr>
<td>$w_c$</td>
<td>Characteristic crack width</td>
</tr>
<tr>
<td>$\Delta D$</td>
<td>Deviation in toughness</td>
</tr>
<tr>
<td>$\Delta P$</td>
<td>Deviation in load</td>
</tr>
<tr>
<td>$\varepsilon_{c3}$</td>
<td>Linear elastic strain limit</td>
</tr>
<tr>
<td>$\varepsilon_{c3u}$</td>
<td>Ultimate strain limit</td>
</tr>
<tr>
<td>$\varepsilon_{ct,ax}$</td>
<td>Linear elastic tensile strain limit</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Density</td>
</tr>
<tr>
<td>$\phi_f$</td>
<td>Fibre diameter</td>
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**Greek Symbols**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
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<tr>
<td>$\varepsilon_{c3}$</td>
<td>Linear elastic strain limit</td>
<td>-</td>
</tr>
<tr>
<td>$\varepsilon_{c3u}$</td>
<td>Ultimate strain limit</td>
<td>-</td>
</tr>
<tr>
<td>$\varepsilon_{ct,ax}$</td>
<td>Linear elastic tensile strain limit</td>
<td>-</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Density</td>
<td>kg/m</td>
</tr>
<tr>
<td>$\phi_f$</td>
<td>Fibre diameter</td>
<td>mm</td>
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