Understanding
Steel Fibre Reinforced Concrete: Dramix®

Guidance to comprehending an extraordinary material
By Gerhard Vitt, Bekaert
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Table of content

Disclaimer ......................................................................................................... 2
Gerhard Vitt ....................................................................................................... 3
Symbols and Abbreviations ................................................................................. 6
Understanding Steel Fibre Reinforced Concrete ..................................................... 7

1 Introduction ................................................................................................... 7

2 Steel Fibres and Basic Material Properties of Steel Fibre Concrete .................. 7
   2.1 Steel fibres ............................................................................................. 8
   2.2 Forms of delivery ..................................................................................... 9
      2.2.1 Loose Steel Fibres ........................................................................... 9
      2.2.2 Glued Steel Fibres .......................................................................... 10
      2.2.3 Dramix® Booster ............................................................................ 11
      2.2.4 Zinc Coated Steel Fibres ................................................................. 12
   2.3 Concrete composition ............................................................................... 12
   2.4 Post crack strength .................................................................................. 14
   2.5 Scatter .................................................................................................. 19
      2.5.1 Beam test ...................................................................................... 21
      2.5.2 Plate tests .................................................................................... 21
      2.5.3 η-factor ......................................................................................... 22
   2.6 Performance class concept ........................................................................ 23
   2.7 Toughness and Ductility ......................................................................... 24
   2.8 Cracking ................................................................................................ 25
   2.9 Fatigue .................................................................................................. 27
   2.10 Impact ................................................................................................... 28
   2.11 Long term performance ....................................................................... 30
   2.12 Durability ............................................................................................. 31
   2.13 Fire resistance ....................................................................................... 34
   2.14 Electrical conductivity and resistivity ..................................................... 35

3 Safety ........................................................................................................... 36
   3.1 Required level of safety ........................................................................... 36
   3.2 Robustness and ductility of structures ..................................................... 37
   3.3 Safety Concept ....................................................................................... 38

4 Quality Control .............................................................................................. 39
   4.1 Fibre quality ........................................................................................... 40
   4.2 Concrete Quality ..................................................................................... 41
      4.2.1 Control of post crack strength ......................................................... 41
      4.2.2 Process control ............................................................................... 42
   4.3 Fibre Orientation .................................................................................... 43
5 SFRC in Flexion
5.1 Difference between section and system properties .......................................................... 44
  5.1.1 Statically determinate system – cantilever .................................................................. 44
  5.1.2 Statically indeterminate system – slab on ground ...................................................... 46
  5.1.3 Effect of additional reinforcement .................................................................................. 47
  5.1.4 Effect of compressive forces .......................................................................................... 48
  5.1.5 How to identify suitable systems .................................................................................. 48
5.2 Design approaches ............................................................................................................... 53
  5.2.1 Stress-strain relation ...................................................................................................... 54
  5.2.2 Stress-crack opening relation ......................................................................................... 56
  5.2.3 Calculation of Section Forces .......................................................................................... 57
5.3 Examples ............................................................................................................................... 59
  5.3.1 Strip foundations and foundation beams ...................................................................... 59
  5.3.2 Industrial floors and pavements .................................................................................. 60
  5.3.3 Floors on Piles ................................................................................................................. 63
  5.3.4 Under water concrete slab .............................................................................................. 65
  5.3.5 Rafts and foundation slabs in housing ......................................................................... 66
  5.3.6 Wall ................................................................................................................................. 68
  5.3.7 Pipes ............................................................................................................................... 69
  5.3.8 Segmental Lining ............................................................................................................. 71

6 SFRC in Shear ............................................................................................................................ 73
  6.1 Section based approach ..................................................................................................... 74
  6.2 Shear .................................................................................................................................. 74
  6.3 Punching shear .................................................................................................................... 75
  6.4 Examples ............................................................................................................................... 76
  6.4.1 Prefabricated Lintels ....................................................................................................... 76
  6.4.2 Pre-Stressed Beams and Roof decks .......................................................................... 78
  6.4.3 Coupling Beams .............................................................................................................. 79

7 Combined Reinforcement: 1 + 1 = 3 ..................................................................................... 80
  7.1 Load bearing capacity ........................................................................................................ 81
  7.2 Crack Width .......................................................................................................................... 82
  7.3 Examples ............................................................................................................................... 84
  7.3.1 Repair of an industrial floor .......................................................................................... 84
  7.3.2 Industrial floor as secondary barrier ............................................................................. 85
  7.3.3 Load-bearing joint free industrial floor ....................................................................... 86
  7.3.4 Raft and Walls in SLS ..................................................................................................... 87
  7.3.5 Raft in SLS and ULS ........................................................................................................ 87
  7.3.6 Columns in high-rise building ....................................................................................... 88
  7.3.7 Concrete face of a rock-filled dam ............................................................................... 89
  7.3.8 Oceanographic Park ....................................................................................................... 90

8 Outlook ..................................................................................................................................... 90

9 Reference .................................................................................................................................. 91
**Symbols and Abbreviations**

### Latin upper case letters

- **A_{ct}** area of concrete in tension just before cracking
- **A_s** area of reinforcement
- **F_{cr}** minimum crushing load
- **F_u** ultimate load
- **M_{Ed}** design value of occurring bending moment
- **M_{Ed}** design bending moment in ULS
- **N_{Ed}** design value of occurring axial force
- **V_{Ed}** design value of occurring shear force
- **Z** section modulus
- **X_{Ed}** design value of X
- **X_{Kd}** characteristic value of X

### Latin lower case letters

- **d** fibre diameter
- **e_{hd}** design values of horizontal pressure
- **f_{c1t,28}** first crack concrete tensile strength at 28 days
- **f_{c2t,28}** post crack concrete tensile strength at 28 days
- **f_{ct,ef}** concrete tensile strength effective at cracking
- **f_{l1}** 1st crack flexural strength
- **f_{l2}** post crack flexural strength
- **f_{l2,d}** design value of post crack flexural strength
- **f_{res}** residual post crack strength
- **f_{eq}** equivalent post crack strength
- **f_{res,1}** average residual flexural strength of the steel fibre reinforced concrete at cracking
- **f_y** yield strength of reinforcement steel
- **h** height
- **k_c•k_p** coefficients allowing for the actual stress distribution in the section
- **l** fibre length
- **l/d** ratio fibre length over fibre diameter (aspect ratio)
- **m** bending moment per unit width
- **m_{l,1}** 1st crack moment
- **m_{Ed}** elastic bending moment in a section
- **m_{pl}** plastic bending moment in a section
- **m_{pl,sys}** plastic bending moment in a system
- **n** number of samples or specimens
- **n** axial force per unit width
- **p** water pressure
- **q_{cr,1}** load leading to 1st yield line
- **q_{cr,2}** load leading to 2nd yield line
- **q_{cr,3}** load leading to 3rd yield line
- **q_{pl}** load leading to final yield line pattern
- **w** crack width / crack opening
- **W_{cr}** crack width in reinforced concrete
- **W_{comb}** crack with in combined reinforced concrete

### Greek letters

- **γ_m** partial factor for resistance (material safety factor)
- **γ_f** material safety factor for steel fibre concrete
- **δ** deflection
- **Δ_h** difference in height
- **Ø** diameter
- **ε_{ct}** strain in concrete tensile zone
- **κ** conversion factor post crack flexural / post crack tensile strength
- **η** conversion factor specimen / structure
- **σ** stress in the reinforcement
- **σ_{l,II}** stress permitted in the reinforcement immediately after formation of the crack

### Abbreviations

- **FPC** factory production control
- **ITT** initial type testing
- **QPC** Quality Performance Concrete
- **RPC** reactive powder concrete
- **SLS** serviceability limit state
- **TBM** tunnel boring machine
- **UHPC** ultra high performance concrete
- **ULS** ultimate limit state
Understanding Steel Fibre Reinforced Concrete

1 Introduction

In the past decades, steel fibre reinforced concrete has evolved from an exotic construction material to a widely used alternative to, or improvement of, conventional concrete, either plain or reinforced. Tremendous research has been done and numerous projects of almost any size have been carried out successfully. Today, steel fibre reinforced concrete is used worldwide in a large range of applications. Despite this great theoretical and practical experience, many people still wonder how it really works. Simply transferring design approaches for plain or reinforced concrete to steel fibre reinforced concrete will hardly lead to success. Steel fibre reinforced concrete is different to reinforced concrete. Therefore, other methods and procedures have to be used for its assessment and understanding.

This brochure aims to shed light on this “well known strange material”. It is meant to explain:
- the principal properties of steel fibres and steel fibre reinforced concrete
- the typical design cases which have to be considered
- the difference between section and system properties
- why some applications can and others can not be designed with steel fibre concrete,
- how suitable applications can be identified
- what this means for the designer and his design.

Examples are given for the most common applications.

Basically it is aimed at engineers, architects, specifiers and all other readers who would like to get an insight into the basics of designing steel fibre concrete and use it successfully.

It is aimed at all those trying to understand steel fibre concrete.

2 Steel Fibres and Basic Material Properties of Steel Fibre Concrete

Unlike traditional steel reinforcement, steel fibres are a discontinuous, 3-dimensionally oriented, isotropic or anisotropic reinforcement - once mixed into the concrete. A variety of fibre types is available. They can be made from different materials, with different shapes and different sizes.

However, their effect on concrete properties varies to the same extent. Therefore steel fibre reinforced concrete shall never be simplified as a “concrete with steel fibres”. On the contrary, it must be seen as a material, which is composed from an appropriate concrete composition, a suitable fibre type and the corresponding amount of fibres to meet the given requirements. Once all conditions are taken into account it may be called a “steel fibre concrete.”
In most if not all countries, the ready-mix industry is able to deliver steel fibre concretes which are fit for the intended use. However, material properties have to be defined in a specification text that was worked out specifically for a project. Therefore, the specifier carries special responsibility to detail the required material properties in a way which still allows competition, but avoids underperforming “alternatives”.

Having sufficient background information about steel fibres and their effect in concrete will help in the preparation of technically correct and economically attractive specifications.

### 2.1 Steel fibres

Steel fibres may be divided into five groups [1], based on their method of manufacture:

- **Group I**: cold-drawn wire
- **Group II**: cut sheet
- **Group III**: melt extracted
- **Group IV**: shaved cold drawn wire
- **Group V**: milled from blocks

As a matter of fact, the vast majority of steel fibres used today belong to group I. Fibres of group II and IV are still used in considerable quantities. Fibres from group III and V play a less important role on the global scale. The reason may be found in the performance fibres of group I provide to concrete.

The following considerations will therefore focus on cold-drawn wire fibres.

Like any reinforcement, it is important to provide as much reinforcement in a section as needed. But comparing fibres on the basis of dosage alone does not lead to the right conclusions. The Reason is that different factors influence fibre performance:

- **Shape**: straight, hooked, undulated, crimped, twisted, coned …
- **Length**: typically from 30 mm to 60 mm
- **Diameter**: typically from 0.4 mm to a maximum of 1.3 mm
- **Tensile strength**: see chapter 2.3, Concrete composition

For the same type of anchorage, length and diameter have the biggest influence on fibre performance. It can be stated [3] that fibre performance increases by increasing fibre length and decreasing diameter. The reasons are better anchorage (length) and a higher number of fibres per kg (diameter). The so-called l/d-ratio, the ratio of fibre length and diameter, by itself gives a good estimate of fibre performance. Table 1 provides an indicative overview of the relationship.
Table 1: indicative relation between fibre geometry and performance (hooked fibres, developed length = fibre length +2 mm)

<table>
<thead>
<tr>
<th>Fibre Type</th>
<th>Length l [mm]</th>
<th>Diameter d [mm]</th>
<th>~l/d-ratio [-]</th>
<th>Fibres [~ pcs./kg]</th>
<th>Wire [~ m/kg]</th>
<th>Performance [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>50/1,3</td>
<td>50</td>
<td>1.30</td>
<td>38</td>
<td>1800</td>
<td>90</td>
<td>lower</td>
</tr>
<tr>
<td>RL-45/50-BN</td>
<td>50</td>
<td>1.05</td>
<td>45</td>
<td>2800</td>
<td>140</td>
<td></td>
</tr>
<tr>
<td>ZC 610</td>
<td>60</td>
<td>1.05</td>
<td>60</td>
<td>2300</td>
<td>140</td>
<td></td>
</tr>
<tr>
<td>RC-65/60-BN</td>
<td>60</td>
<td>0.90</td>
<td>65</td>
<td>3200</td>
<td>190</td>
<td></td>
</tr>
<tr>
<td>RC-80/60-BN</td>
<td>60</td>
<td>0.75</td>
<td>80</td>
<td>4600</td>
<td>275</td>
<td>higher</td>
</tr>
</tbody>
</table>

Varying the type of anchorage will change the shape of the steel fibre concrete’s load-deflection curve, see 2.4. Nevertheless, it will not change the underlying principle of a high l/d-ratio being preferred.

The recently introduced product standard EN 14889-1 [1] for steel fibres allows to obtain a quick overview of fibre properties and even performance from the compulsory CE-label. For further details, please refer to chapter 4.1.

2.2 Forms of delivery

The thinner and the longer a fibre is, the better it will perform in concrete (chapter 2.1). It is sometimes stated that long thin, high aspect ratio fibres are more prone to balling in concrete and that it is not possible to mix them at higher dosages. This cannot be taken as granted, as proved by the remarkable track record of high l/d Dramix® fibres.

There is, of course, a certain truth in that statement. High l/d-ratios correspond with a huge amount of single fibres and a large total wire length (Table 1). All this wire has to be placed somewhere in the concrete matrix and thus requires more attention to the mix design. But comparing 40 kg/m³ easy-mix fibres (low l/d) with 40kg high-perform fibres (high l/d) would mean comparing apples to oranges. Instead of said 40 kg /m³ easy-mix fibres, one may only need 20 kg/m³ or even 15kg/m³ of high-perform fibres to achieve the same performance, depending on the types being compared. Given this context, things can to be seen from a different angle.

Special systems have been developed to avoid balling and to allow for optimal fibre distribution.

The right form of delivery, together with a suitable concrete composition (chapter 2.3) and mixing procedure, will allow even 100 kg/m³ of high-performing fibres like the Dramix® RC-80/60-BN to be effectively mixed into concrete.

2.2.1 Loose Steel Fibres

“Loose” is the typical form of delivery for fibres with a relatively low aspect ratio, say l/d ≤ 50 (Picture 1). Balling or other difficulties related to workability may hardly be expected. An example is the easy-mix type RL-45/50-BN, offering a length of 50 mm at a nominal l/d of 45. However, due to the low aspect ratio, their performance level may be considered as basic.
For higher aspect ratios, say $l/d \geq 60$, additional methods should be applied in order to easily and effectively add the fibres to the concrete. One option, the use of blower blast equipment can be suggested (Picture 2) in such a case.

Nevertheless, users in the field tend to avoid blowing loose fibres with an aspect ratio higher than ~70 for a number of practical reasons, e.g. a persisting risk of balling. In addition, it needs more time and effort compared to other forms of delivery. Also, blowing fibres is equal to adding fibres but not equal to dosing, at least with the ordinary equipment available. Dosing requires an exact determination of how many kilograms of fibres have been introduced into the concrete.

**2.2.2 Glued Steel Fibres**

In order to avoid the potential for balling related to adding loose fibres with a high (and thus better performing) aspect ratio, glued fibre technology has been developed. After adding glued Dramix® steel fibres to concrete, the bundles spread evenly on the “macro”-level. Continued mixing lets the individual fibres separate quickly so that they can then homogeneously distribute on the “micro”-level.
Glued steel fibres can be added right out of the bag directly into a mixing truck or central mixer, or indirectly via a conveyor belt. Automatic dosing equipment is also available (chapter 4.2.2).

### 2.2.3 Dramix® Booster

The booster system is a further development of the degradable Dramix®-bags used in the past. The degradable 20kg bags have been replaced by a continuous chain of small paper bags, containing 250g fibres each. Even high aspect fibres can be put in the bag loosely, thanks to the small bags causing a similar effect to bundling and gluing (chapter 2.2.2).

By counting the number of individual 250g bags, the fibres can be dosed automatically by the Booster. The small paper bags dissolve in the concrete completely. Batching plants are the optimum location for the booster to add and dose steel fibres in an automated, safe and cost efficient way.
2.2.4 Zinc Coated Steel Fibres

When steel fibres are galvanized, the concrete surface will be kept clear of rust stains due to the corrosion of any visible fibres, or parts of fibres, at the surface. This is of particular importance when a corrosion free surface is needed, for example with precast elements. A potential problem with galvanized steel fibres is that the zinc coating when coming into contact with cement paste can cause the formation of hydrogen gas.

Picture 5: spongy and porous surface around zinc coated steel fibres without inhibitor (left), smooth surface around Dramix® Green fibres with inhibitor (right)

Zinc coated Dramix® steel fibres, also called Dramix® Green, are delivered as glued fibre bundles. The glue contains an environmentally friendly and unique inhibitor, which protects the zinc coating against the chemical reaction with the alkaline constituents of the concrete. Without protection, concrete quality and performance would be substantially lower due to:

- a spectrum of fibres visible at concrete surface
- a porous concrete surface because of bubbles
- a loss of durability because of gas formation
- a lack of bond leading to reduced post crack strength

Bekaert has prepared more detailed information about the effect of an alkaline concrete environment on zinc coated steel fibres and the positive effects of suitable protection systems.

2.3 Concrete composition

It was already indicated that steel fibre reinforced concrete is a distinct concrete in its own right. Simply mixing steel fibres into any concrete will most probably not utilize all the positive effects fibres can provide to concrete. Depending on the type and amount of fibres, adjustments to the concrete mix design may need to be made. Examples such as:

- increasing the mortar content,
- increasing the content of fines,
- adjusting the grading curve,
- using plasticizer and / or super plasticizer

may need to be considered.
Another important subject is the correct choice of steel fibre tensile strength. This is due to the mechanism by which fibres reinforce concrete: the fibres need to be slowly pulled out of the concrete matrix. The anchorage system, typically end hooks, should deform and provide sustained resistance to pullout rather than prevent it. As the failure strain of steel fibre wire is relatively low, breaking of fibres must be avoided, otherwise the post crack strength (2.4) and thus toughness and ductility (2.7) reduce too quickly, see Table 2.

Using end-hooked fibres made from normal strength steel wire of about 1000 MPa – 1400 MPa tensile strength, the switch from ductile to brittle failure is to be expected for a concrete strength going higher than 50-60 MPa. Concrete strength, however, does not relate to the nominal but to the actual strength of the concrete. In addition, the type of anchorage can significantly affect performance. End hooks have proven to provide excellent performance both in the initial and final stages of fibre pull-out.

<table>
<thead>
<tr>
<th>Fibre concrete</th>
<th>Single fibre</th>
</tr>
</thead>
<tbody>
<tr>
<td>normal strength fibre - normal strength concrete</td>
<td><img src="chart1.png" alt="Chart" /></td>
</tr>
<tr>
<td>normal strength fibre - high strength concrete</td>
<td><img src="chart2.png" alt="Chart" /></td>
</tr>
<tr>
<td>fibre with fixed ends</td>
<td><img src="chart3.png" alt="Chart" /></td>
</tr>
</tbody>
</table>

**Table 2: effect of fibre tensile strength and type of anchorage**

The embrittlement of steel fibre reinforced concrete at higher concrete strengths has been known for a long time. To overcome this high strength fibres have been developed. The shape usually does not vary from the normal strength fibres but tensile strength is above 2500 MPa. These fibres are used in high strength concrete and even ultra high strength concretes.
For concrete qualities such as UHPC or RPC it is possible to use high strength fibres without any additional mechanical anchorage. The extreme bond between wire and concrete matrix allows the use of straight steel wire fibres. Their tensile strength is typically 2000 MPa or higher. As the large majority of steel fibres today are used in normal strength concrete normal strength steel fibres with some form of mechanical anchorage, provide the optimal solution. Bekaert is prepared to give theoretical and practical recommendations on how to choose the right fibre type and how to optimize concrete composition and mixing procedure.

2.4 Post crack strength

One of the major characteristics of steel fibre reinforced concrete is its ability to transfer stresses (read: forces) over a cracked section. This is unlike plain concrete, which loses its entire load bearing capacity once it’s cracked. Just like conventional (mesh or bar) reinforcing steel, steel fibres act as reinforcement. Steel fibres pick up stresses from the earliest stages of cracking. Less crack energy is therefore released. The longer fibres are and the greater the number (fibre count) available in a concrete matrix, the higher will be the amount of fibres bridging a crack.
The majority of steel fibre concretes used today show a behaviour, which is classified as strain-softening, see Figure 3. The wider the crack opening gets, the lower the load carrying capacity gets. This is particularly the case in direct tension but it is also observed in pure bending. In the latter case it is possible to produce a bending-hardening behaviour – which is hardly ever the case in direct tension.

Post crack tensile strength can be converted from post crack flexural strength by means of conversion factors. These factors depend on the assumed stress strain behaviour of fibre reinforced concrete and the way the testing is performed.

The level and the shape of the post crack strength curve basically depend on:

- fibre class in accordance with chapter 2.1.
- number of fibres in a crack
- type of anchorage
- fibre length
- fibre orientation
- concrete matrix and strength
- fibre tensile strength vs. concrete tensile strength

As cold-drawn wire fibres have become a standard, other fibre types will not be considered in the following. In terms of anchorage hooked ends are the most popular, flattened end, undulated fibres, those with a cone at the ends and combinations thereof all being found.

**Figure 4: effect of dosage**
- higher dosage
- lower dosage

**Figure 5: effect of l/d-ratio**
- higher l/d
- lower l/d

**Figure 6: effect of fibre length**
- longer fiber
- shorter fiber

**Figure 7: effect of anchorage**
- hook
- undulated

**Figure 8: effect of concrete**
- better concrete
- worse concrete

**Figure 9: effect of wire strength**
- sufficient
- insufficient
In order to evaluate the effect of all these parameters on the performance of a steel fibre concrete, deflection controlled beam or plate tests have become the standard testing procedure, see Figure 10. A number of standards, guidelines and recommendations are available. Representing others it shall be mentioned:

- EN 14651 Europe three-point-bending test, notched [7]
- JSCE-SF4 Japan four-point-bending test [8]
- NBN B 15-238 Belgium four-point-bending test [9]
- CUR 35 Netherlands four-point-bending test [10]
- SIA 162/6 Switzerland four-point-bending test, panel tests [11]
- DAFStb-guideline Germany four-point-bending test [27]
- DBV-recommendation Germany four-point-bending test [28]
- ÖVBB-guideline Austria four-point-bending test [26]
- ASTM C1609 USA four point bending test [10]
- ASTM C 1550 – 02 USA round determinate panel test [11]
- UNI 11039 Italy four-point-bending test, notched [13]

**Figure 10: outline of beam tests and round determinate panel test (front / top view)**

Once the load-deflection curve has been derived, it can be reported according to the relevant test method. This may either be equivalent values (Figure 11), equivalent values taking into account a part of the fracture energy (Figure 12) or residual values. Those may either be post crack flexural or post crack tensile strength values. For each test setup, a different relation (plot) between deflection and crack opening may be found.

Equivalent values are interpreted as the mean post crack strength over a certain area under the load deflection curve. Some regulations require the subtraction of the fracture energy, which is related to the first crack strength of the concrete, some others don’t. As a simplification, a triangle is often used for approximation.
In recent times, residual values have been introduced. At certain deflections (read: crack openings) the corresponding loads are read from the curve. These values are absolute values and do not represent some kind of average, as is the case for equivalent values, see Figure 13.

For the same fibre concrete, residual values derived at larger crack openings are smaller than the corresponding equivalent values. However, residual values better represent the slope of the load-deflection curve and thus the effects of strain softening. Furthermore, the fracture energy assigned to the concrete is of less or even no importance any more. Despite equivalent post crack strength for two different fibre types being the same, their residual values may differ significantly, see Figure 14.

Bekaert’s concrete lab is familiar with all relevant testing standards for fibre concrete. Experts are prepared to provide technical support or carry out tests.
2.5 Scatter

Repeating a deflection-controlled test with a set of specimens taken from the same concrete will give different results. Even the mean values of a series of samples may show some variation, depending on the actual number of samples in a set. The variation or standard deviation within a set will be reduced by increasing the number of samples. This is a fundamental principle of statistics.

Figure 15: distribution of mean post crack strength for two different fibre concretes depending on the number of samples

Therefore, one should always be careful when comparing two different steel fibre concretes or two different fibre types in the same concrete, as explained by Figure 15. In the case of a small number of samples (full lines), two actually completely different materials (red and blue) could show the same post crack strength. It is possible to mistakenly conclude that the two different materials perform the same! This is due to an overlap of the individual distribution functions (grey hatched area). By increasing the number of samples contained in a set (dashed lines), variation becomes smaller. If the tested materials are indeed different, the overlap will disappear and the individually tested mean post crack strengths will be closer to the “real” mean post crack strengths.

It also becomes obvious that comparing individual values of two different steel fibre concretes does not make any sense at all.
A part of this scatter is due to the nature of the material, but another part is due to the way the specimens are made and the test is set-up. Important factors influencing scatter are:

- Number of fibres
- Size of Specimen
- Ratio max. fibre length / max. aggregate size
- Concrete mixing
- Casting of specimens
- Quality of test equipment
- Experience of the laboratory

Most factors may be kept quite under control if testing is carried out in an experienced concrete lab by experienced people. However, not all scatter can be eliminated by improving concrete mix design and testing procedure. A significant portion of overall scatter is due to the size of the area in tension for the nominated test method [13].

As long as the test specimen's area in tension is approximate of the same size as that of a structure, there is no contradiction. Nevertheless, scatter decreases in proportion to the size of the area in tension. Without additional measures, the actual safety of a structure with a larger tensile zone would be underestimated enormously if design values were based on small test specimens alone.

![Figure 16: coefficient of variation vs. area in tension](image)

This is why [15] suggests to use an upgrade factor if design values are based on characteristic values derived from standard beams. In [25], [26] and [27] an upgrade factor (chapter 2.5.3) has been introduced while [23] applies an application related coefficient of variation in order to determine the characteristic value from a set of test beams. Another option would be testing larger sample sizes, say a minimum of 15 beams (chapter 2.5.1) or 5 round determinate panels (chapter 2.5.2) as also suggested in [15]. However, this would lead to more expensive testing, at least concerning the required number of beams. The round determinate panels, with a larger tensile area, may offer a cost advantage in regards to testing if compared to the beams.
One should always be critical in comparing individual results and in defining performance classes (chapter 2.6) unless the effect of scatter is fully understood, especially if those classes are based on characteristic values.

With Bekaert you can rely on a large database and the experience from its own concrete laboratory specialized on testing all kinds of steel fibre concrete specimens over many years.

2.5.1 Beam test

Typically, a set of 6 beams, tested in accordance with the test methods listed in 2.4, shows a coefficient of variation between 15% and 40%. The size of the tension zone arrives at ~ 170cm² to 200cm².

Tests published in [13] resulted in a maximum scatter of 35%, whereby the mean scatter was 22%. This confirms practical experience of about 20%-30% scatter to be expected in standard beam tests. The exact type of test method may have an influence as well, e.g. equivalent values versus residual values or notched versus un-notched beams.

2.5.2 Plate tests

- Round determinate panel (“RDP”)

According to the standard test method [11], a round panel is placed on three rotating supports so that a statically determinate system is formed. In the cracked state three cracks should be observed, each approximately bisecting a pair of supports. Due to the specimen geometry, the round determinate panel test shows a coefficient of variation typically below 20%. The size of the tension zone is ~ 800cm².

Tests published in [13] resulted in a maximum scatter of 15%, whereby the mean scatter was 10%. This is significantly lower than for the standard beams.

Material properties to be applied in a stress-strain or stress-crack width relation may be calculated from a round determinate panel test. The position of the cracks (read: yield lines) has to be taken into account for the evaluation.
• EN 14488-5 panel (“EFNARC panel”)

EN 14488-5 [16] originates from the EFNARC panel test [17] and is used to determine the energy absorption of a fibre concrete. A square panel is placed on a square steel frame so that a statically indeterminate system is formed. The number of cracks (yield line) depends on the steel fibre concrete performance. This is why - unlike beams or round determinate panel tests - system characteristics instead of material properties are determined. The indeterminate panel test according EN 14488-5 simulates the behaviour of a shotcrete lining. Therefore, it is to be used for such applications. With 4 cracks, the tension zone is about 1,000 cm², resulting in a coefficient of variation similar to the one found for the round determinate panel.

2.5.3 \( \eta \)-factor

Scatter has a major effect on the safety of structures. As outlined above, the scatter of steel fibre reinforced concrete reduces in proportion to the size of the area in tension. For that reason, characteristic values or design values for steel fibre concrete vary with the type of structure.

An approach introduced in EN 1990 [21] is to introduce a conversion factor as follows:

\[
X_d = \eta \cdot \frac{X_k}{\gamma_m}
\]

With:
- \( X_d \) design value
- \( X_k \) characteristic value
- \( \gamma_m \) partial factor for resistance (material safety factor)
- \( \eta \) conversion factor specimen / structure – may be incorporated into \( X_k \)

Assuming a factor \( \eta \) bigger than 1.0, the characteristic values derived from standard beam tests (larger variation) may be adjusted for the design of structures with larger tension zone (lower variation). The factor \( \eta \) is the EN 1990 equivalent to the upgrade factor, which was suggested in [15]. As an alternative, the derivation of the characteristic value from beam test results may be linked to a certain application by applying related conversion factors.

Recent design guidelines such as [22], [23], [25], [26] or [27] consider these approaches. Depending on the overall approach, \( \eta \cdot X_k \) for the design of slab type structures may be almost equivalent to the population mean of a series of standard test beams [27].

The \( \eta \)-factor is a very suitable tool to align modern safety concepts and the variation of test specimens with the actual behaviour of real steel fibre concrete structures and the introduction of performance classes (chapter 2.6).
2.6 Performance class concept

In some countries, performance classes have been introduced to classify the post crack strength of steel fibre concrete. To the established classification of concrete compressive strength, further criteria can be added to describe the post crack strength.

Typically, this is achieved with two additional values, relating to both the serviceability limit state (SLS) and the ultimate limit state (ULS) respectively. The example shown in Table 3 is based on the use of mean equivalent tensile strengths derived from beam tests. It is equally possible to define performance classes incorporating any combination of characteristic/mean, equivalent/residual and post crack tensile/flexural values determined from beam/plate tests. In the case of shotcrete the use of energy absorption values derived from plate tests is also an option.

<table>
<thead>
<tr>
<th>example of performance class</th>
<th>C30/37 F1,2/1,0</th>
<th>according to [28]</th>
</tr>
</thead>
<tbody>
<tr>
<td>characteristic compressive strength (cylinder) in MPa</td>
<td>C30/37 F1,2/1,0</td>
<td>relates to EN 206-1 [29]</td>
</tr>
<tr>
<td>characteristic compressive strength (cube) in MPa</td>
<td>C30/37 F1,2/1,0</td>
<td></td>
</tr>
<tr>
<td>mean equivalent post crack tensile strength (SLS) in MPa</td>
<td>C30/37 F1,2/1,0</td>
<td>derived from four point bending test</td>
</tr>
<tr>
<td>mean equivalent post crack tensile strength (ULS) in MPa</td>
<td>C30/37 F1,2/1,0</td>
<td></td>
</tr>
</tbody>
</table>

Table 3: example of performance class denomination according to [28]

In all cases, performance classes allow the designer to concentrate on the actual design of a structure, not spending additional time in determining the right type and amount of fibres to achieve the required performance. The latter responsibility falls to any concrete supplier offering performance classes.

The introduction of performance classes requires capable laboratories with suitable test equipment and on going quality control during all stages of concrete processing. Together with the approach of the “$\eta$-factor” (chapter 2.5.3) or the upgrading factor (chapter 2.5), performance classes effectively support the concept of characteristic values.

Bekaert is familiar with existing performance classes and is prepared to give advice on how they can be achieved and used. Consequently, a large number of ready-mix organizations have developed this relatively new concept together with Bekaert.
2.7 Toughness and Ductility

Steel fibre reinforced concrete has traditionally been associated with the properties of toughness and ductility. The reason is its ability to combine load-bearing capacity with large deformation capacity. Very often toughness is expressed by equivalent strength or energy absorption.

In both tunnelling and mining, steel fibre reinforced concrete has become the state of the art for the construction of shotcrete linings. Energy absorption classes have been introduced [17], and are being used for the design of tunnel linings. The required energy absorption curves are derived by integrating the load-deflection curves of sprayed steel fibre concrete panels (Figure 17, chapter 2.5.2).

![Figure 17: typical load-deflection curve of steel fiber reinforced sprayed concrete panel used for determination of energy absorption classes, according [76]](image)

It is important to point out that testing toughness using statically indeterminate plate tests is equal to testing system performance. Toughness testing does not mean determining material properties:
The statically indeterminate plate test [16] is taken to represent a shotcrete lining which is anchored to the ground (see chapter 2.5.2). It cannot be compared with the results from either statically determinate beam or panel tests.

For further information, Bekaert has published dedicated papers and literature. ([18], [34], [76])
2.8 Cracking

It is a well-known fact that steel fibres have a very positive effect on cracking and crack propagation. The distance between steel fibres is much smaller than typical spacing for reinforcing bars. Unlike reinforced concrete, fibres are distributed throughout the whole section. Hence, there is no concrete cover without reinforcement. Furthermore, stresses in the root of a crack can be picked up quicker. This is why crack propagation and crack patterns change when compared to plain or even reinforced concrete:

As steel fibres usually bridge cracks at a non-perpendicular angle they will already be deforming and picking up load at small crack widths. Local friction is increased and thus compressive stresses parallel to the crack surface are induced.

![Figure 18: secondary cracking in steel fibre concrete [37]](secondary_cracking)

As a consequence, the associated tensile stresses perpendicular to the crack can lead to secondary cracking (Figure 18). Those cracks may be compared to cracks in reinforced concrete, which can be found in the zone directly around re-bar. With steel fibre concrete, secondary cracking can be observed over the whole cracked section. Subsequently cracks become more curved. Fragmentation, translation and multiplication can be identified [37], see Figure 19. Resistance to intruding substances, especially liquids, is substantially increased [38]. Aggregate interlock and friction are also enhanced [39].

![Figure 19: steel fibres influencing crack pattern [37]](crack_pattern)
However, due to the strain softening behaviour of most steel fibre concretes used today calculation of crack width is only possible under certain conditions (see chapter 7.2). Crack widths originating from restraint deformation can definitely not be assessed by means of calculation.

But even if one cannot always calculate crack widths, in many cases it is reasonable to benefit from the ability of steel fibre concrete to influence cracking. Applications such as industrial floors and shotcrete do not require crack width calculation in most cases. Crack width design is replaced by testing of system properties [16], experience and defined conditions, which can be related to the application [40]. Even for segmental linings, subject to extreme spalling and bursting stresses, steel fibre concrete can be superior to reinforced concrete because of its unique reinforcing properties [19].

As an example, so called “jointless” steel fibre reinforced industrial floors can be reviewed. These floors do not have any saw cut joints but special steel profiles at large distances, say of approximately 30 m to 40 m, see 5.3.2). They have become the preferred type of floor in many countries, especially when talking about distribution centres and warehouses.

Jointless floors require an overall design and detailing approach in order to reduce restraint to a minimum. Despite cracking not being avoided in all cases, it is well known from experience that crack widths will most likely be in the range from about 0.3 mm to 0.5 mm – if occurring at all. This presumes a high performing steel fibre reinforced concrete, accurate design and detailing [40]. The preferred fibre length for this application is 60mm whereby the aspect ratio l/d is 65 or higher. Shorter fibre length may be compensated by a higher aspect ratio (l/d) due to the large total wire length provided per m³; see also Table 1.

Serviceability of these floors is still excellent, even in cracked state, as their load bearing capacity does not decline ([35], [36]) and load transfer over the unplanned joints (read: eventual cracks) is maintained [39]. Unlike sawcut floors, spalling of joints cannot occur as a principle. Special joint profiles are used to protect the day-joints and to assure load transfer. In case cracks larger than 0.5 mm should occur repair is possible.
Nevertheless, many other applications have to be designed for a certain crack width, especially structural applications. Relying on experience and accurate detailing alone may not always be sufficient and subsequently not meet the requirements. For instance, a design crack width may need to be determined to prevent corrosion of re-bar or aesthetic damage, ensure water tightness or protect the environment from dangerous substances. In these cases, steel fibres can contribute significantly to support the effect of traditional reinforcement (chapter 7.2).

Both experience and research have proven that steel fibres are an excellent choice “when a tight crack matters”. May it be steel fibre concrete alone or in combination with traditional rebar: the structure will surely benefit from it.

Bekaert has contributed to a number of research projects and has gained practical experience in a variety of projects.

2.9 Fatigue

For an increased fatigue resistance the presence of many steel fibres in concrete is definitely advantageous. Especially positive influence on root stresses, the dowelling of the full section and the improved crack pattern (see chapter 2.8) all seem to enhance resistance to fatigue loading. Very positive experience is available from both testing [42] and practical experience, such as roundabouts, highways (autobahn), slab tracks or machine foundations.

The dosage and the performance of a particular fibre both have a strong influence on the fatigue limit. A substantial increase in fatigue performance for fibre reinforced concrete can only be expected where an increase in strength after cracking is achieved. This increase in strength can either be obtained by strain hardening steel fibre concrete in bending or by suitable support conditions, as given for ground supported or other statically indeterminate systems (see chapters 3.2 and 5). Both possibilities provide a sufficient amount of performing steel fibres.

However, no design approach is available today, which can be used to calculate the effect of steel fibres. A good summary of literature is presented in [41], sub-task 5.3. Existing experience or application related testing might be used instead.

![Picture 7: example of steel fibre reinforced roundabout](image)
2.10 Impact

Impact resistance is enormously increased by the addition of steel fibres. A number of tests are available to simulate distributed or single point, hard or soft impact, at high or low velocity, taking into account static or dynamic loading in single or even multiple cycles. Impact could, for instance, be from bullets (hard, single point impact, high velocity), shock waves (single cycle, distributed), or falling blocks (single point, single or multiple cycles, low velocity), Figure 21.

In all cases, the addition of steel fibres significantly improves the resistance of the plain concrete. Figure 20 demonstrates once again the importance of choosing the right type of fibre, see also chapter 2.1. The results have been derived by using a 4.54 kg hammer and a drop of 457 mm according to the procedure described in [43].

![Figure 20: number of blows on concrete specimens according [43]](image)

The combination of bar reinforcement and fibres is especially beneficial in the case of shock wave loading (shelters for military or industrial applications).

Benefits of steel fibre reinforced concrete are

- reduced spalling and scabbing of panels
- increased energy absorption
- increased number of blows up to first crack
- increased number of blows up to failure
- increased resistance to cavitations

Also in this application, an increased post crack performance of the steel fibre concrete in static beam/panel testing leads to an increase in impact resistance. Larger amounts of high performing steel fibres are preferred in order to provide the required dense fibre network, see chapter 2.1. The improvement may even reach orders of magnitudes compared to concretes without fibre, also confirmed by Figure 20.
Suitable applications may be driven piles, segmental linings, shelters, bunkers, containment structures, intake structures, spillways, stilling basins or other protective elements.

In [47] an impressive case is documented where a containment building was designed for an explosion which could potentially originate from a chemical production process within. The structure was designed and reinforced with a combination of traditional steel bars and high performing steel fibres. Only a few days after production had started, the production process ran out of control and a heavy explosion occurred. The production line was completely destroyed but the structure was almost undamaged. The new production could be set up again in the same building.
2.11 Long term performance

While a number of design guidelines introduce long-term performance factors ([26], [27], [41]), others don’t ([20], [25], [57]). Apparently, the origin of those long-term factors dates back to investigations on steel fibre reinforced concrete pipes in 1986 [48]. An additional long-term factor has since been introduced in the DBV-recommendations [45], [46] and subsequently taken up by others. At least this approach does not seem to be on the unsafe side and comes with relatively low safety factors for the material itself.

Sometimes results of relaxation tests are translated into long-term factors. But as [41], subtask 7.2 concludes this may not always be right:

“Relaxation tests on laboratory-sized notched specimens used for the flexural toughness tests and larger unnotched beams have demonstrated that the load drop in the post-cracking regime under constant crack width can be of the order of 40%. However, subsequent reloading led to the recovery of practically the complete load-carrying capacity before the relaxation.”

Due to the nature of relaxation, a deflection-controlled test may not be the best choice to determine long-term performance of steel fibre concrete. The same can be said for conventionally reinforced concrete.

Recent testing on specimens with combined reinforcement [49] came to a similar conclusion as stated in [41], subtask 7.2 for the ultimate limit state.

Table 4: comparison of moment-strain relation for specimens under short term and long term loading according to [49]
Within the context of long term performance, creep deformation in the serviceability limit state should also be considered. In-house tests [50], run in the Bekaert concrete laboratory and tests initiated by the Austrian society for concrete and construction technology (OVBB, [51]) demonstrated a low increase in deflections at serviceability stress levels for steel fibres. Despite no specimen with reinforced concrete being investigated, the test results for macro synthetic fibre reinforced concrete does at least provide a strong basis for comparison between the performances of these two fibre types in terms of creep.

The tests were carried out with standard test beams subject to a permanent load representing about 50% of the residual load achieved at a defined deflection. The load-level was chosen to simulate the actually occurring stress level in serviceability limit state.

![Deflection versus time curve for macro synthetic and steel fibre (RC-65/35-BN) concrete beams under flexural load according [50]](image)

**2.12 Durability**

In chapter 2.8, 2.9 and 2.10 the effect of fibres on Cracking, Fatigue and Impact is explained. Improving concrete performance within these fields automatically increases the durability of a related application.

On the other hand, using steel as a construction material also raises corrosion resistance as an important issue. In regards to steel fibre reinforced concrete, cold-drawn steel wire fibres (group I, see 2.1) do benefit from a variety of mechanisms to counter corrosion:

- no effect on the electrical resistivity of concrete at normal dosage (chapter 2.14)
- discontinuous reinforcement
- smooth and dense surface
- small dimensions
- thus relatively low electrochemical potential
- no concrete spalling
Due to these properties corrosion resistance of group I fibres is typically higher when compared to ordinary reinforcing steel bars. This should not be seen as a given for fibres of groups II - IV.

In the case of uncracked concrete, group I fibres are less sensitive to corrosion than reinforcing steel bars, even in a seawater environment. Only fibres situated directly at the surface will show corrosion. This corrosion does not penetrate into the concrete and does not cause concrete spalling [52], [53] and should therefore be considered as an aesthetic problem only.

Using stainless or zinc-coated fibres may be an option to prevent surface corrosion. (Zinc coated fibres need to be protected against hydrogen formation in fresh concrete, see also chapter 2.2.4. Due to cost reasons, stainless steel fibres are of limited practical importance.)

In cracked concrete, however, corrosion appears to be linked to the actual crack width and the type of exposure [41], [54]. Similar to rebar, carbonation is less critical than the presence of seawater or de-icing salt. The presence of moisture is significantly more detrimental than a dry atmosphere.

While no significant corrosion potential could be found in cracks up to 0.5 mm due to carbonation, in a chlorine environment the non critical crack width was reduced to 0.2 mm [41]. At 0.5 mm, indications of slight corrosion were found. Comparable conclusions have also been reported in other publications [55], whereby [56] challenges the maximum crack width approach. [54] suggests smaller crack widths than [41].

When discussing severe exposure conditions, it must not be forgotten that most applications are associated with ordinary environmental conditions. Furthermore, the severe effects of a chlorine environment are also well known for traditional reinforcement. In the case of reinforced concrete, there is only one layer of reinforcement. Once this layer is penetrated by chlorides, all the reinforcement is being affected at the same time. As steel fibres are distributed throughout the whole section, the chances are much less that all fibres are located within a layer of critical chloride concentration.

For the design of structures, durability requires either suitable environmental conditions or small crack widths. The latter is exactly one of the main reasons why steel fibres are used in many applications. It appears obvious to use steel fibres to protect traditional reinforcement from corrosion, especially in a severe environment. The maximum design crack width, of course, depends on the type of exposure and type of fibre [25].
Table 5: Recommended choice of steel fibres according [25]

<table>
<thead>
<tr>
<th>Type of concrete</th>
<th>Type of fibre</th>
<th>A: wire fibre</th>
<th>B: sheet fibre</th>
<th>C: other fibre / 1: low carbon</th>
<th>2: high carbon</th>
<th>3: stainless</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>A3-B3-C1</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
</tr>
<tr>
<td></td>
<td>A1-B1-C1 ST</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
</tr>
<tr>
<td></td>
<td>A1-B1-C1</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
</tr>
<tr>
<td></td>
<td>A2-B2-C2 ST</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
</tr>
<tr>
<td></td>
<td>A2-B2-C2</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
</tr>
<tr>
<td>C2</td>
<td>A3-B3-C1</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
</tr>
<tr>
<td></td>
<td>A1-B1-C1 ST</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
</tr>
<tr>
<td></td>
<td>A1-B1-C1</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
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</tr>
<tr>
<td></td>
<td>A2-B2-C1 ST</td>
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</tr>
<tr>
<td></td>
<td>A2-B2-C2</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
</tr>
<tr>
<td>C3</td>
<td>A3-B3-C1</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
</tr>
<tr>
<td></td>
<td>A1-B1-C1 ST</td>
<td>YES</td>
<td>YES</td>
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</tr>
<tr>
<td></td>
<td>A1-B1-C1</td>
<td>YES</td>
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<td>YES</td>
</tr>
<tr>
<td></td>
<td>A3-B3-C1 ST</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
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<td>YES</td>
</tr>
<tr>
<td></td>
<td>A3-B3-C2</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
</tr>
</tbody>
</table>

Some design guidelines for steel fibre concrete relate environmental conditions to allowed crack widths [27], [57]. Special provisions like coatings, failure layers or oversized post crack strength may also be considered in the case of a severe environment [54], [57].

<table>
<thead>
<tr>
<th>Exposure class (*)</th>
<th>steel fibres</th>
<th>steel fibres + reinforcement</th>
<th>steel fibres + pre-tensioning</th>
<th>post-tensioning</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>(****)</td>
<td>(****)</td>
<td>0.2 mm</td>
<td>0.2 mm</td>
</tr>
<tr>
<td>2</td>
<td>0.3 mm</td>
<td>0.3 mm</td>
<td>0.2 mm</td>
<td>decompression</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(****): for exposure class 1, crack width has no influence on durability and the limit could be relaxed or deleted unless there are other reasons for its inclusion.

Table 6: criteria for crack width according [57]

Long-term experience exists for a large number of applications, even in harsh environments. Examples are, exterior pavements, tunnel segments or sewer pipes. Million of square meters of industrial floors, numerous tunnel linings, uncounted load bearing foundation slabs or precast elements tell their own tale and may provide further guidance on durability aspects.
2.13 Fire resistance

Despite the fact that concrete does not burn, it does not always cope with fire very well. Moisture contained within the concrete needs to escape from the matrix as steam. If steam is produced faster than it can escape, explosive spalling is likely to occur. Even if spalling only affects areas close to the surface, it can still expose reinforcement quickly and directly to the fire. The load carrying capacity of the structure can be quickly lost. Collapse is more than likely, because the tensile strength of steel drastically reduces with increasing temperature [58].

Steel fibres, when added at normal dosages, do not significantly increase the thermal conductivity of concrete, which is similar to their effect on electrical conductivity (chapter 2.14). Unlike steel reinforcement, the fibres are distributed throughout the whole section. The discontinuous fibres are not concentrated in a thin layer close to the surface and thus are not all close to heat source. Consequently, the risk of losing the load-bearing capacity of all steel fibres at the same time is negligible.

Recommendations such as [25] or [28] have introduced the concept of a failure layer, the thickness of which is dependant on both temperature and the required time of fire resistance. Also large-scale fire testing, as in the case of steel fibre concrete reinforced steel-decks, is able to establish the duration of fire resistance [59] that can be achieved with steel fibre reinforced concrete. In steel-deck application, steel fibres allow the redistribution of those stresses once taken by the steel deck – before it was heated.

However, steel fibres cannot protect concrete from thermal deterioration. There are indications that the inclusion of steel fibres reduces spalling due to thermal shock and thermal gradient. But steel fibres should only be seen as an additional measure which may lead to improved steam escape. If improved fire resistance is required, the addition of polypropylene fibres, a suitable choice of aggregates and concrete composition should be the first choice [60].

Micro-polypropylene fibres, used for fire protection of concrete, have performed very well in many tests all around the world. With Duomix® Fire, Bekaert offers a well-suited product for this application and is prepared to provide further information.
2.14 Electrical conductivity and resistivity

Electrical conductivity and resistivity of concrete play an important role in those applications, where stray currents may disrupt signals (guided vehicles) or create unsafe conditions (explosion protection). Electrical resistivity is also important where reinforced concrete will be exposed to corrosive conditions, as corrosion supporting or inducing currents will flow more easily in low resistivity concrete.

Keeping the amount of cement paste low, using good quality pozzolans and avoiding the introduction of ionized inorganic salts will result in lower conductivity and higher resistivity [61]. Temperature effects are important as well, whereby higher temperatures give lower resistivity.

Adding steel fibres will change neither concrete resistivity nor conductivity. The fibres are discontinuous and are spread three-dimensionally throughout the whole concrete.

The critical concentration at which an electrically continuous three-dimensional network is formed cannot be reached with steel fibre concrete. (Note: this may be different for the very special case of Slurry Infiltrated Concrete, SIFCON, which contains 800 kg/m³ steel fibres or even more.). In a worst-case scenario for steel fibre concrete, fine concrete containing 80 kg/m³ Dramix ZC 40/0.40 was compared to the same concrete without fibres. Based on the commonly used fibre types and dosages; the chosen amount of this specific steel fibre corresponds to an extremely dense fibre network and thus countless potential contacts between steel fibres. In practical terms, this fibre concrete may certainly be seen as a very exceptional mix.

Despite these severe conditions, no significant increase in conductivity could be found when compared to the plain concrete specimen. Furthermore, investigations based on RADAR did not find higher shielding after fibres had been added.

In addition, tests have been carried out on concrete with extraordinary high dosage rates such as 400 kg/m³. The aim was to maximize conductivity, without considering economic constraints. Even though relatively high macroscopic conductivity was obtained, the value reached is still far below the conductivity of metal sheets or bars.

Thus, ordinary steel fibre concrete is not a suitable material for conductive concrete, EMP-applications or radiation shielding. For the same reason, there is no need to worry about steel fibres affecting concrete conductivity or resistivity in applications such as tunnel linings, industrial floors or other applications subject to a corrosive environment. The results of the above-described tests have been reported in [62].
3 Safety

3.1 Required level of safety

Safety is nothing more than the inverse of risk. Therefore, the required level of safety basically depends on the potential risk to the health or lives of people. Environmental or financial considerations may also have some influence. This is why the required safety level for bridges is regularly higher than for a typical industrial floor. Building Codes usually distinguish between:

- critical infrastructure (e.g. hospital)
- ordinary structures (e.g. office building)
- minor structures (e.g. industrial floor)

In regards to steel fibre concrete, a lack of design standards is still the status quo in most countries. Even for minor applications design standards do not exist in a number of countries, sometimes not even simple recommendations are available. The situation differs from application to application and from country to country. Nevertheless, major achievements have been made [20], [25], [27], [30] or [84].

If no recommendations are available, approvals, design by testing or sound engineering judgment may be considered, depending on the actual application. In the latter case, all technical and legal aspects need to be taken into account. The engineer should have sufficient knowledge and experience with regards to both the structure and to steel fibre concrete.

More general design recommendations often suggest material safety factors $\gamma_f$ of 1.25 to 1.5 for steel fibre concrete. Any safety factor refers to a defined quality control regime and/or specific design rules within the scope of the design recommendation: For instance, [27] suggests $\gamma_f = 1.25$ while referring to general design of concrete structures. However, it only allows the use of ready-mixed concrete in combination with tight internal and external process control. Specific design rules have to be followed and other factors in addition to $\gamma_f$ have to be applied. As defined in the scope, [27] is not applicable for the design of industrial floors where adding fibres on site is still an accepted practice.

A case dependent approach was chosen in [23] for the specific application of cellar walls. Here the material safety factor to be applied depends on the method of fibre dosing and on the chosen quality control regime. Using advanced dosing equipment (chapter 4.2.2) together with a defined quality control system results in more economic fibre dosages while meeting the required safety level.

<table>
<thead>
<tr>
<th>$\gamma_f$</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>manual dosing in plant</td>
</tr>
<tr>
<td>1.3</td>
<td>automatic dosing in plant, internal quality control of homogeneity and performance, tested by batching plant and Bekaert, additional cooperation with certifying body SECO</td>
</tr>
</tbody>
</table>

Table 7: safety factors for cellar wall design according to [23], taken from [24]
Examples of safety factors always have to be seen in the context of the overall design approach and must not be taken on a standalone basis. Bekaert can look back on a long track record of all kinds of applications, supporting your correct choice of the right safety concept.

### 3.2 Robustness and ductility of structures

One major principle in construction is to create robust structures. Robustness is directly linked to the ductility of a structure. This may be summarized in the statement.

“The first crack must never be the last crack.”

Of course, there is no rule without exception, but the vast majority of structures must not collapse once the first crack has formed. On the contrary, failure should occur in a predicted way so that response measures can be taken in time. This requires an increase in the load bearing capacity whilst deformation and cracking increase. Table 8 gives a principle overview of failure modes.

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<td>F</td>
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<td>F</td>
</tr>
</tbody>
</table>

(a) brittle, unpredicted  
(b) ductile, unpredicted  
(c) almost brittle = unpredictable  
(d) ductile, predicted

**Table 8: overview of failure modes, based on load-deflection relation**

In reinforced concrete design, so-called minimum reinforcement is foreseen to provide required ductility, see Table 8 (d). Typically, it has to be designed to take up the forces arising in a section right after cracking. Apart from special detailing provisions, usually no further considerations are necessary [33].

At this point, many designers ask themselves how a strain softening material like steel fibre reinforced concrete (see 2.4) can be used for the design of robust and ductile structures. The answer to this question can be found in the distinction between section and system properties:

In statically determinate systems, the section properties are equal to the system properties.

As soon as one single section fails, the whole structure fails. For typical SFRC in bending this means that the first crack is indeed the last crack.
But depending on the static system and even the type of loading, SFRC shows a load bearing capacity which can far exceed the performance achieved in statically determinate beam tests. Each section of a structure still behaves like a section in a beam test. Based on the section behaviour of SFRC itself, structural ductility cannot be shown. The difference is the overall interaction of structure, support conditions, loading and material.

By using steel fibre reinforced concrete in the right way for the right applications, robustness and ductility are no contradiction to a strain softening section behaviour. Eurocode 2, 5.10.1 [33] already foresees several options to avoid brittle failure for pre-stressed members and structures. Some of them may be transferred to steel fibre concrete as well:

“(6) Brittle failure should be avoided by one or more of the following methods: Method A: Provide minimum reinforcement in accordance with 9.2.1. […] Method E: Ensure that if failure were to occur due to either an increase of load or a reduction of pre-stress under the frequent combination of actions, cracking would occur before the ultimate capacity would be exceeded, taking account of moment redistribution due to cracking effects.”

Chapter 5.1 explains the difference between a section and a system approach in more detail.

### 3.3 Safety Concept

A safety concept always comprises much more than “just” safety factors and design formula or design approaches. Appropriate documentation, rigid specification texts, sufficient quality control, both on a material and building level, site supervision and proper execution in accordance with a plan are important elements in any safety concept.

Chapter 4 focuses on quality control of fibres and fibre concrete. For the other elements mentioned, Bekaert can assist you with sample texts or even site support.
4 Quality Control

Quality control is an essential element in providing safe and durable structures. Despite steel fibres being delivered as a component of the concrete, quality control must not be neglected by adopting the attitude “It’s all in the concrete – I can’t check it!”. To the contrary, in most cases a few simple steps are sufficient to check if the relevant requirements are met.

With regards to steel fibre reinforced concrete, it is of utmost importance to know if the

- fibres have the required quality certificates
- right type of fibres were used
- right amount of fibres were used
- fibres are distributed uniformly
- required performance can be provided by the concrete mix

Depending on the application and also on the concrete producers and end users familiarity with steel fibre concrete, all or only some checks have to be made at a reasonable frequency. This may also depend on the total volume of steel fibre reinforced concrete supplied by a producer.

The quality control regime may be focused on either material or process control. Material control primarily focuses on controlling the material properties of the delivered product. Continuous testing of post crack strength would be a suitable option for this approach. Typically, a set of specimens (beam, round determinate panel etc.) per certain volume of steel fibre concrete has to be tested.

A process control based approach, however, would rather focus on controlling all steps when making steel fibre reinforced concrete rather than testing what has come out afterwards. Testing post crack strength would, of course, still be essential but the frequency could be reduced if there confidence in the reliability of the process. Once post crack strength has been determined (initial type testing) and providing that neither concrete composition nor fibre type or amount are changed, control of fibre content and fibre distribution will ensure the performance. Whenever important parameters in the process change, initial type testing has to be repeated. Confirmation testing may be done at certain intervals. As concrete compressive strength is still to be tested, additional control possibilities are given. A drop in compressive strength will also indicate a drop of post crack strength.

One of the major requirements of the process control concept is certified material and equipment. The concept was, for instance, introduced in [27] with full compatibility to [29].

Bekaert has also introduced a quality control concept called Dramix® QPC – Quality Performance Concrete, which is especially suitable for ready-mix applications and markets where no standards for quality control of steel fibre concrete exist yet.
4.1 Fibre quality

Performance is always linked to the quality control regime applied to a product. Steel fibres are no exception here. It may be risky to rely on the promises of a manufacturer alone. On the contrary, international standards for production and quality control of steel fibres for concrete reinforcement are available, such as EN 14889-1 [1] and ASTM A 820/A 820M-04 [2]. Depending on the individual country, local approvals may amend or replace them. ISO 9001 certified plants and quality control according to the requirements of the mentioned standards and approvals are to be seen as state of the art – and the minimum one should accept today.

In particular the CE-label, related to EN 14889-1, allows a quick but significant estimate of fibre properties. Important information, such as geometric data, tensile strength and l/d-ratio has to be shown on that label (see also chapter 2.1). Furthermore, a minimum dosage has to be declared which is related to a required minimum performance in a reference concrete [5], [6]. This allows an estimate of the performance level of a fibre without the need for further investigation. Examples are given in Table 9.

<table>
<thead>
<tr>
<th>Dramix® Type</th>
<th>Length l [mm]</th>
<th>Diameter d [mm]</th>
<th>~l/d-ratio [-]</th>
<th>CE-minimum dosage [kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>RL-45/50-BN</td>
<td>50</td>
<td>1.05</td>
<td>45</td>
<td>20</td>
</tr>
<tr>
<td>RC-65/60-BN</td>
<td>60</td>
<td>0.90</td>
<td>60</td>
<td>15</td>
</tr>
<tr>
<td>RC-80/60-BN</td>
<td>60</td>
<td>0.75</td>
<td>80</td>
<td>10</td>
</tr>
</tbody>
</table>

Table 9: minimum fibre dosage for CE-labelling according [1], [5], [6].

According to EN 14889-1, “structural use of fibres is where the addition of fibres is designed to contribute to the load bearing capacity of a concrete element.” But it also specifies steel fibres for other than structural uses. Therefore, one should take care not to mistake fibres with declaration of conformity for fibres with an EC-certificate of conformity. The actual difference between those products may be significant.

<table>
<thead>
<tr>
<th>Responsibilities for quality control [1]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural use System “1”</td>
</tr>
<tr>
<td>Initial type testing</td>
</tr>
<tr>
<td>Initial inspection of factory and of FPC</td>
</tr>
<tr>
<td>Continuous surveillance</td>
</tr>
<tr>
<td>Assessment and approval of FPC</td>
</tr>
<tr>
<td>EC Certificate of conformity</td>
</tr>
<tr>
<td>EC Declaration of conformity</td>
</tr>
</tbody>
</table>

Table 10: responsibilities for quality control, system “1” and system “3” according [1]
In order to avoid any potential risk, steel fibre should always comply with system “1”, testified by an EC certificate of conformity. Dramix® steel fibres are produced under system “1”. They are fit for structural use.

4.2 Concrete Quality

Using high quality steel fibres is the first step towards high quality steel fibre concrete. However, it takes more effort than applying the normal quality control for ordinary concrete and simply adding fibres to the mix. Nowadays many ready-mix suppliers are offering steel fibre concrete – the more advanced of them even with a performance warranty for the specific steel fibre concrete properties. Two different approaches may be distinguished. The main principles are explained in the following chapters.

4.2.1 Control of post crack strength

The great benefit of testing post crack strength is first hand information of the material properties of the concrete actually used. For quality control testing, a specimen size with low scatter is preferred. Otherwise, more specimens or wider tolerances will be the consequence. Despite beam tests, as described in chapter 2.5.1, being regarded as the standard performance evaluation method today, the intrinsic scatter of this test method always requires a sufficient amount of specimens [15].

As this is relatively costly and time consuming, the Round Determinate Panel test RDP as described in chapter 2.5.2 may be considered as a suitable alternative. Investigations on the correlation between EN 14651 beams [7] and RDP [11] have already been made. It should be noted that the tensile zone of the application should at least have the same size as the tensile zone of the RDP.

Nevertheless, experienced people and suitable machinery are still required. Testing cannot be carried out on site and results can only be expected after 28 days, as the concrete needs to harden.

To give an example, control of post crack strength was applied in an existing approval for load bearing foundation slabs [31], dating back to 2000. The control regime foresaw a set of three beams per either 500 m³ of steel fibre concrete or per 6 production days. As this was one of the first general approvals for load bearing structures made from steel fibre concrete, a rather conservative approach was applied. It has turned out that practice prefers to try and get around regulations, which it regards as too strict and conservative.

This is why according to [26] confirmation testing of post crack strength may be replaced by testing both first crack flexural strength and fibre content. If first crack flexural strength is higher than 90% of the initial type testing, if concrete composition has not changed and if fibre content is sufficient, post crack strength is assumed sufficient as well. Background is the correlation of first crack strength and post crack strength, which is valid if the same type of fibre is used in the same type of concrete.

Other regulations [27] allow for a similar approach, further developing this idea into a full process control approach (chapter 4.2.2).
4.2.2 Process control

Gaining more and more experience with quality performance control has resulted in another way of surveying steel fibre concrete being established. Focus is shifted from final product control to controlling the entire production process. The basic principle is the idea that steel fibre concrete properties will not change if neither concrete matrix properties nor type, nor amount and nor distribution of steel fibres are changed.

Based on an initial type testing ITT of hardened steel fibre concrete specimens, future control will check if the

- constituent materials are still the same (type and strength class of cement, nature of aggregates, admixtures etc.),
- concrete composition has not changed (compare mix design with ITT),
- dosing equipment works properly (check regularly, especially fibre dosing equipment),
- steel fibres are still the same (type of fibre, supplier and certificate of conformity),
- correct type of steel fibre is mixed into the concrete (compare with fibre type used in ITT),
- correct amount of steel fibres is mixed into the concrete (washout test),
- steel fibres are distributed homogeneously (washout test at certain intervals per delivery unit).

Whenever there is a major modification in the production process, ITT has to be repeated and used as new reference for future supply. Independent from that, a periodic conformation test as a repetition of the ITT should be made at least once a year or in case of doubt [26], [27].

Lower safety factors may even be introduced for a product with improved process control (see chapter 3, [22], [23]).

It is evident that the process control based approach asks for automatic dosing equipment. Specialized solutions like Dramix® VIBRO and Dramix® BOOSTER are available on the market (Picture 10). Hence, “dosing” should not been confused with “adding” fibres. Conveyor belt or blast blower equipments are transportation devices. Dosing always requires measuring volume or weight.

Picture 10: Dramix® VIBRO (left) and Dramix® BOOSTER (right)
The great benefits of the process control based approach are “real-time” results on site, simple procedures and acceptable levels of effort. It is obvious that practice is going to prefer the process control approach to continuous post crack strength testing.

4.3 Fibre Orientation

Fibre orientation always occurs at the interface of mould and concrete. It may also be influenced by concrete consistency and composition, flow distance and method of casting [32]. Usually orientation is not considered for cast in situ slabs or comparable structures where orientation is beneficial for the load bearing capacity. This is also the case for shotcrete linings or certain thin precast elements.

In addition, the effect may be used to improve product properties by influencing fibre orientation due to the casting method. As an example, the production of segments for tunnel linings is a case in point.

In some cases, special provisions like an orientation factor may be considered in the design. Calibration on real applications is recommended. In [57], an orientation factor has been included in the shear design, which was derived from real scale beam tests. Furthermore, it is suggested that “the designer should indicate whether the orientation is important and then specify either acceptable method for the compaction and the casting of SFRC and/or indicate which methods must not be used.”

For the majority of applications today, fibre orientation seems to be negligible or even beneficial.
5 SFRC in Flexion

Without any doubt, the ability of steel fibre concrete to carry bending moments even in cracked sections has been the key to its success. Most successful applications utilize its post crack flexural strength to reinforce plain concrete or to replace or amend ordinary reinforcing steel. A number of design approaches have been published. Some are of rather general nature while others refer to dedicated applications. Therefore, it is of great importance to understand the design principles of steel fibre concrete in flexure.

Special consideration always has to be made for tensile stresses originating from restrained deformation. This may either be by controlling those stresses (basic conditions, constructional processing, minimum performance etc.), by choosing uncritical applications (permanent compression zone) or by using combined reinforcement (see 7).

5.1 Difference between section and system properties

Designing steel fibre concrete means understanding the difference between section properties and system properties. Otherwise, the design might be either uneconomic or unsafe. The latter, in particular, should not occur.

The following chapters will explain the basic principle in order to allow the identification of suitable systems (see 5.1.5) by means of engineering judgment or calculation. Two simple examples, cantilever and slab on ground, will be used to demonstrate the difference. Furthermore, it is explained, how the required minimum performance to achieve ductile system behaviour may be derived for a clamped beam or slab.

5.1.1 Statically determinate system – cantilever

A cantilever is a system where the section capacity equals the system capacity. The weakest section will govern the load bearing capacity. A comparison between plain, reinforced and steel fibre reinforced concrete is shown in Table 11.
Table 11: comparison section capacity versus system capacity, statically determined

It is obvious that strain softening steel fibre concrete alone cannot be used for statically determinate systems. As soon as the first crack strength is exceeded due to the loading, the cracked section is no longer capable of resisting the acting bending moment. Consequently, the cantilever will collapse: its first crack is its last crack. Of course, this applies to plain concrete as well. Reinforced concrete will resist the loading, providing the required minimum reinforcement has been used. (Otherwise, it will collapse as well.)

Exceptions might be possible for certain structures of minor importance and minor risk, compare [31]. Strain hardening steel fibre concrete would be an alternative to reinforced concrete. However, such steel fibre concretes usually result in significant cost. Due to that, this option is hardly considered current practice.

5.1.2 Statically indeterminate system – slab on ground

One of the standard applications today is steel fibre concrete slabs on ground, such as industrial floors or even foundation slabs. These are statically indeterminate systems, which allow redistribution of loads and stresses, both in the ground and in the slab. Despite the section properties of steel fibre concrete being strain softening, the system “slab on ground” shows a different behaviour when compared to a statically determinate system.

<table>
<thead>
<tr>
<th>Material</th>
<th>Section:</th>
<th>System:</th>
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<tbody>
<tr>
<td>Plain concrete</td>
<td><img src="image" alt="Diagram" /></td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>Steel fibre concrete</td>
<td><img src="image" alt="Diagram" /></td>
<td><img src="image" alt="Diagram" /></td>
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<tr>
<td>Reinforced concrete</td>
<td><img src="image" alt="Diagram" /></td>
<td><img src="image" alt="Diagram" /></td>
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</tbody>
</table>

**Table 12: comparison section capacity versus system capacity, statically undetermined**

By choosing a statically indeterminate, plane system, strain-softening material can provide sufficient ductility and robustness. Important research (e.g. [35], [36]) has been carried out to investigate the performance of steel fibre reinforced concrete slabs. It was shown that the load leading to the first crack is much lower than the load leading to the last crack (read: yield line).
Once the first section exceeds first crack strength, the steel fibres will still transfer a certain part of the section forces. Thus, stresses can be redistributed within slab and ground. More and more cracks (read: yield lines) can form now so that both bearing capacity and deformation capacity increase. An increase in load-bearing capacity by the magnitude of three times first crack load or even above has been reported [35], [36].

Compared to reinforced concrete, the steel fibre reinforced slab behaves very similarly: robust and ductile. Load bearing capacity increases with deformation until the limit is reached. Plain concrete, however, will also benefit from the stress redistribution. But unlike steel fibre concrete, it has to rely on the ground alone and thus the increase in load bearing capacity is very much lower. Furthermore, the integrity of the slab is lost completely once cracked. In practical terms this means that the slab might have to be removed.

Bekaert has contributed to those investigations. Detailed information and dedicated documents can be provided upon request.

5.1.3 Effect of additional reinforcement

Adding reinforcement to steel fibre concrete – or vice versa – will increase the overall load bearing capacity. Providing sufficient amounts of reinforcement are used, section properties will shift from strain softening to strain hardening. By doing so, the minimum ductility criterion is fulfilled and further consideration concerning that may be omitted.

In [57], the following formula is proposed for calculating the minimum reinforcement $A_s$ in order to obtain controlled crack formation.

$$A_s = \left( k_c \times k \times k_p \times f_{ct,ef} - 0,45 \times f_{rm,1} \right) \times A_{ct} / \sigma_{s,ii}$$

Equation 1

where:
- $A_s$ area of reinforcement within tensile zone (mm²)
- $k_c, k, k_p$ coefficients allowing for the actual stress distribution in the section
- $f_{ct,ef}$ the tensile strength of the concrete effective at the time when the cracks may first be expected to occur (N/mm²)
- $f_{rm,1}$ average residual flexural strength of the steel fibre reinforced concrete at the moment when a crack is expected to occur (N/mm²)
- $A_{ct}$ area of concrete within tensile zone just before cracking (mm²)
- $\sigma_{s,ii}$ maximum stress permitted in the reinforcement immediately after formation of the crack (N/mm²).

This may be taken equal to the yield strength of the reinforcement ($f_{y}$). However, a lower value may be needed to satisfy the crack width limits.

If $A_s$ is smaller than zero only steel fibres are necessary, according to [57]. Other design approaches have derived different equations but all have in common designing minimum reinforcement based on the stresses released when cracking. Differences can be found in the way actual stresses are taken into account.
More information about combined reinforcement is provided in chapter 7.
5.1.4 Effect of compressive forces

In general, compressive forces are beneficial for the load bearing capacity of steel fibre concrete. Whenever a compressive zone is permanently present in a section, crack width calculation and thus proof of structural ductility is possible. The coefficient $k_p$ in Equation 1 allows for compressive stresses, for instance. Important applications where compressive forces are taken into account are pre-cast or in-situ tunnel linings, walls and pre-cast containment structures.

5.1.5 How to identify suitable systems

Depending on the individual static system, type and amount of steel fibres, compressive forces or traditional reinforcement, the failure mode may be either ductile or brittle. As can be seen from Figure 23, high performing steel fibre concrete provides sufficient load bearing capacity to reinforce a point-supported slab. Nevertheless, in this particular static system a combination with little reinforcement along the support grid or moderate pre-stressing forces along the support grid results in a more distinctive ductile failure mode.

![Graph showing load versus deflection for different reinforcement configurations](image)

Figure 23: point-supported slab according to [69] with 40 kg/m³ Dramix RC-80/60-BN (Equation 1 ≈ 0), $d = 15$ cm, 5 m x 5 m

This leads to the question, how to identify suitable applications if neither practical experience nor test results are available.

One possibility is to compare the load required to achieve the first crack (read: yield line) with the maximum load after all yield lines have formed. The procedure shall be illustrated by the example of a single span beam with a clamp support on both sides. Step by step the applied line load is increased so that concrete flexural strength is exceeded and yield lines form. For each step, the corresponding load is calculated which is required to achieve the next yield line, see Table 13.

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<table>
<thead>
<tr>
<th>q</th>
<th>$f_l$</th>
<th>$f_{sl}$</th>
<th>W</th>
<th>1st crack strength</th>
</tr>
</thead>
<tbody>
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<td></td>
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<td></td>
<td>post crack strength</td>
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<td></td>
<td>section modulus</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(flexural strength)</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>$m_{cr,1}$</th>
<th>$q_{cr,1} = 12\cdot W\cdot f_l/l^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$m_{cr,1} = W\cdot f_l$</td>
<td></td>
</tr>
<tr>
<td>$q_{cr,1}$</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$m_{cr,2}$</th>
<th>$q_{cr,2} = 8\cdot W\cdot (f_l + 1/2\cdot f_{sl})/l^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$m_{cr,2} = m_{cr,1}$</td>
<td></td>
</tr>
<tr>
<td>$m_{pl} = W\cdot f_{sl}$</td>
<td></td>
</tr>
<tr>
<td>$q_{cr,2}$</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$m_{cr,3}$</th>
<th>$q_{cr,3} = 8\cdot W\cdot (f_l + f_{sl})/l^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$m_{cr,3} = m_{cr,1}$</td>
<td></td>
</tr>
<tr>
<td>$m_{pl} = W\cdot f_{sl}$</td>
<td></td>
</tr>
<tr>
<td>$q_{cr,3}$</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$m_{pl}$</th>
<th>$q_{pl} = 16\cdot W\cdot f_{sl}/l^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q_{pl}$</td>
<td></td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>$m_{pl}$</th>
<th>$q_{pl} = 16\cdot W\cdot f_{sl}/l^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q_{pl}$</td>
<td></td>
</tr>
</tbody>
</table>

Table 13: derivation of plastic load bearing capacity for single span beam with clamp support on both sides

Based on the calculated yield loads, the ratio of elastic ($q_{cr,1}$) and plastic ($q_{pl}$) load bearing capacity can be calculated. Hence, each individual ratio for a sufficient number of steps should be increasing if compared to the previous one or at least remain on a constant level. For the example of a single span beam with clamp support on both sides one will get:
structural ductility in function of \( \frac{f_{\text{fl,II}}}{f_{\text{fl}}} \)

\[ \frac{f_{\text{fl,II}}}{f_{\text{fl}}} : \]

- 200%:
- 150%:
- 100%:
- 50%:
- 0%

Crack propagation

Actual loading versus first crack load

Figure 24: structural ductility in function of post crack flexural strength versus first crack flexural strength

It becomes obvious that the ratio of post crack flexural strength versus first crack flexural strength should exceed 1.0 in order to achieve a ductile failure mechanism. For ratios lower than 1.0, the system is stabilized by first crack flexural strength. Due to the limited number of yield lines in that example, this may not be acceptable for most applications.

With regards to the nomenclature of Table 8, failure mode could be classified as follows:

- \( f_{\text{fl,II}} / f_{\text{fl}} > 1.0 \): ductile, predicted failure
- \( f_{\text{fl,II}} / f_{\text{fl}} \leq 1.0 \): ductile, unpredicted failure
- \( f_{\text{fl,II}} / f_{\text{fl}} \leq 0.75 \): almost brittle, ≈ unpredictable failure
- \( f_{\text{fl,II}} / f_{\text{fl}} \leq 0.5 \): brittle, unpredicted failure

By replacing the static system by a two-way slab with continuous clamp support along the perimeter the situation changes.
Table 14: derivation of plastic load bearing capacity for two-way slab with continuous clamp support along perimeter

Now failure mode could be classified as follows:

\[
\frac{f_{\text{II}}}{f_t} \leq 0.4: \quad \text{brittle, unpredicted failure} \\
\frac{f_{\text{II}}}{f_t} > 0.4: \quad \text{changes from brittle, unpredicted to ductile, predicted failure}
\]

The same principle may be applied for any other application. As an alternative, a non-linear finite element calculation may be performed instead. However, in the latter case calibration to test results may be required.
• Examples of suitable systems

- ground slabs
- underwater concrete slabs, bolted shotcrete linings
- floor on piles (combined reinforcement)

- walls
- tanks
- pipes and tunnel linings
- shells

- combined reinforcement
- pre-stressed or post tensioned structures

and many others…

Table 15: Examples of suitable systems
• Examples of non-suitable systems for steel fibre only

Table 16: Examples of non-suitable systems

5.2 Design approaches

Once it is known that the static system of a certain structure is suitable for SFRC, the right design approach has to be chosen. From a technical point of view and as a rule of thumb, plastic or non-linear methods are preferred for the ultimate limit state design. From a practical point of view, however, linear-elastic approaches are often used in reality. This may be due to rather limited availability of plastic design approaches or lack of non-linear design software suitable for steel fibre reinforced concrete.

Sometimes, ease of use or a deficiency in background information can result in applying the “second best” design approach.
There is no doubt, of course, that each of the mentioned approaches can be used in principle. Nevertheless, there might be some impact on the technical or economic performance of the result. Strain-softening material properties of steel fibre concrete should be taken into account for the design, at least if the material is to be utilized efficiently.

Stress-strain relations have been developed but there is no such thing as “the stress-strain relation” for steel fibre concrete. On the contrary, many different rules, guidelines and recommendations exist. Despite the core content being not very different (It’s all mechanics!), each single recommendation should only be applied within its full context. Borrowing content from another recommendation should only be done if the overall context is very well understood by the designer.

Bekaert is involved with all major committees on steel fibre concrete design worldwide. Its technical managers can provide specific information if required.

5.2.1 Stress-strain relation

The most common approach for the design of steel fibre concrete is a constitutive law based on a fictitious stress-strain relation. It is fictitious because the steel fibres are pulled out - they are not elongated as steel bars would be. Strain is just used as a vehicle to calculate the equilibrium of internal forces. Otherwise, no link to concrete strain and steel strain (combined reinforcement) could be made.

All required input data for the stress-strain relation is usually derived from beam tests as explained in chapter 2. Evaluation is either based on equivalent or residual flexural strength but normally equivalent or residual tensile strength is applied in the constitutive laws.

Tensile strength values are derived from the corresponding flexural strength by means of conversion factors. These conversion factors are calculated by equating two bending moments. One is the bending moment equal to a linear elastic stress distribution in the section and relates to the post crack flexural strength. The second is the bending moment equal to the applied stress-strain relation. By setting \( f_{t,\|} = \kappa f_{b,\|} \), conversion factor \( \kappa \) can be derived; with \( f_{t,\|} \) = post crack tensile strength and \( f_{b,\|} \) = post crack flexural strength. Figure 25 shows an example for a constant stress strain distribution at large crack opening. See also [57], appendix 1, for more details.

\[
\frac{f_{b,\|}}{f_{t,\|}} = \kappa \cdot \frac{f_{b,\|}}{0.5h} = \kappa = 0.36
\]

Figure 25: derivation of conversion factor for given stress-strain relation
In fact, conversion factors depend on the assumed stress-strain relation [25], [81]. Whenever shape or basis of the stress-strain relation changes, conversion factors change as well (Table 17). Nevertheless, for all design regulations the bending moment derived from back calculation according to the applied stress-strain relation has to be equal to the bending moment derived in the beam test. It has to be a closed loop. Otherwise, either conversion factors or the applied stress-strain relation might have some intrinsic fault.

<table>
<thead>
<tr>
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<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>tensile strength values</td>
<td>equivalent</td>
<td>residual</td>
<td>equivalent</td>
<td>residual</td>
</tr>
<tr>
<td>conversion factor SLS</td>
<td>0.45</td>
<td>0.40</td>
<td>0.45</td>
<td>0.45</td>
</tr>
<tr>
<td>crack width in beam test</td>
<td>~ 0.5 mm</td>
<td>~ 0.5 mm</td>
<td>~ 0.3 mm</td>
<td>0.5 mm</td>
</tr>
<tr>
<td>conversion factor ULS</td>
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<td>0.25</td>
<td>varies</td>
<td>0.37</td>
</tr>
<tr>
<td>crack width in beam test</td>
<td>~ 3.0 mm</td>
<td>~ 3.5 mm</td>
<td>~ 1.8 mm</td>
<td>3.5 mm</td>
</tr>
<tr>
<td>maximum strain in tensile zone</td>
<td>1.0%</td>
<td>2.5%</td>
<td>1.0%</td>
<td>2.5%</td>
</tr>
<tr>
<td>stress-strain relation</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>block as alternative</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>not foreseen</td>
</tr>
<tr>
<td>stress-strain relation</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(dotted line)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>beam type [mm²],</td>
<td>150-150-600</td>
<td>150-150-600</td>
<td>150-125-500</td>
<td>150-125-500</td>
</tr>
<tr>
<td>width-net-height-span</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 17: comparison of different design regulations with regards to constitutive laws

In the case of steel fibre reinforced concrete without additional reinforcement and subject to bending without axial forces, calculation may simply be based on post crack flexural strength, Equation 2. Again, this is due to the closed-loop principle used to determine the conversion factors.

\[ M_d = f_{f,ld} \cdot Z \]

**Equation 2**

where:
- \( M_d \): design bending moment (ultimate limit state)
- \( f_{f,ld} \): design value of post crack flexural strength
- \( Z \): section modulus
Also the maximum crack width in the ultimate limit state needs to be limited. Typically, to those values defined as maximum crack width in the standard beam test. Many common design recommendations, like [25], [28] or [57] convert strain into crack width by applying a geometry dependant conversion.

### 5.2.2 Stress-crack opening relation

Bringing to mind what has been said about the fictitious stress-strain relation, a stress-crack opening relation appears to be the preferred choice when designing steel fibre concrete. Information about the corresponding crack width would then automatically be given as an additional result of calculation. The maximum crack width of the underlying test setup would then not be exceeded.

This would require a method to transfer crack width into strain or otherwise the concrete compression zone or traditional reinforcement could not be calculated.

By defining a certain crack opening as an equivalent to a certain strain, the same strain will always result in the same calculated crack opening. The question this raises is whether a 3.5 mm crack opening is equal to 1.0% or 2.5% or some other value. But whatever the answer is, one always ends up at the same crack width for the same strain, irrespective of how thick the structure is.

Actually, the “correct” value (1.0%, 2.5% or whatever) only becomes important for the design of

- steel fibre concrete sections subject to bending in combination with significant compressive forces
- combined reinforcement

Steel strength is indeed dependent on strain, whereas strain in the concrete compression zone only becomes more important in the case of higher compressive forces. In the case of pure bending of steel fibre concrete, strain in the compressive zone is not critical. Also, as more traditional reinforcement is provided in a section, the less will be the impact of the post crack strength and thus the less influence a potential mistake in the strain-crack width relation will have on the overall result.

In [27] a fixed relation between crack width and strain has been introduced:

\[ w = \varepsilon_{ct} \times 140 \text{ mm} \]

**Equation 3**

where:

- \( w \) : crack opening
- \( \varepsilon_{ct} \) : strain in tensile zone
- 140 mm : fixed value
The fixed value of 140 mm is due to defining a crack opening of 3.5 mm equal to a strain of 2.5% (140 mm = 3.5 mm / 2.5%), see Table 17. Sometimes it is interpreted as the length of the crack, or the height of the tensile zone in the standard beam test as per [27]. Nevertheless, this is coincidence.

During the elaboration of [27], a number of reverse-analysis and additional tests [82] were performed. Based on that it was suggested that a

- scaling factor is not needed if Equation 3 is used;
- maximum steel fibre concrete strain of 2.5% corresponds well enough with the reverse-analysis of combined reinforcement at a maximum steel strain of 2.5%.

In Figure 26 it is shown that normalized bending moment capacity does not depend on the height of a section if compared at the same crack width and fibre segregation is prevented by a suitable concrete composition.

![Figure 26: normalized bending moment - crack width relation for different beam heights and different post crack strength according [82]](image)

### 5.2.3 Calculation of Section Forces

Depending on the application, different methods to calculate the section forces may be preferred. Typically it is distinguished between linear-elastic, plastic and non-linear design. In some specific cases, an elastic-plastic alternative may be applicable.

Most designers are very familiar with the linear-elastic design approach: double loads will result in double stresses and in double deformations. It is most common for the design of plain or reinforced concrete structures. Thus, the same approach is willingly applied to steel fibre concrete also.

Without a doubt, the linear-elastic method is fully applicable to steel fibre concrete design. But due to the nature of the method, system properties can not be taken into account in most cases. This is particularly the case for those structures which are suitable for steel fibre concrete (chapter 5.1). It may be used but it is on the very safe side and thus hardly economical.

On the contrary, the plastic design approach is based on the ductility of steel fibre reinforced concrete and takes both material and system properties into account. Redistribution of bending moments and rotation capacity of the section are key to plastic design. It is also commonly known as yield-line-theory and applied in the ultimate limit state.
A suitable static system is required to achieve both sufficient load-bearing capacity and ductility, see chapter 5.1. Often, existing design formulas for reinforced concrete are applied to steel fibre concrete as well. Hence, confirmation by testing may be advisable. Depending on the application, design formulae already adapted to steel fibre concrete are available [63].

Nevertheless, non-linear design appears to be most suitable for a non-linear material like steel fibre concrete. On the other hand, engineers often do not feel comfortable with this tool. It’s not very common in their day-to-day business, superposition of stresses is no longer possible and the choice of suitable software is very limited. Calibration of non-linear calculations on real test data is often required and a manual check of the results is difficult and often cumbersome.

Plastic design approaches usually give similar results for the load bearing capacity in the ultimate limit state so they are preferred for practical reasons.

In the very special case of elastically supported industrial floors, [83] suggests an elastic-plastic approach. A bending moment calculated according to the linear-elastic method is compared to a bending capacity of the system by taking into account overall dimensions, first crack and post crack strength in the form of two separate factors. Table 18 shows the principle for the three different approaches.

<table>
<thead>
<tr>
<th></th>
<th>elastic-elastic</th>
<th>elastic-plastic</th>
<th>plastic-plastic</th>
</tr>
</thead>
<tbody>
<tr>
<td>bending moment</td>
<td>section based:</td>
<td>section based:</td>
<td>system based:</td>
</tr>
<tr>
<td></td>
<td>( m_{el} )</td>
<td>( m_{el} )</td>
<td>( m_{pl,sys} )</td>
</tr>
<tr>
<td>resistance</td>
<td>section based</td>
<td>system based</td>
<td>system based</td>
</tr>
</tbody>
</table>

Table 18: comparison of design approaches for elastically supported industrial floors

Irrespective of the design approach, the stress-strain relation (see 5.2.1) and the stress-crack width relation (see 5.2.2) are the essential tools to calculate both the load bearing and rotation capacity of a section. System performance equals the performance of either one section (elastic) or of many sections (plastic, elastic-plastic – if applicable).

Due to the fragmented situation concerning general availability of design regulations, legal or administrative conditions and existing practical experience, no design rules shall be enlightened in detail. In Chapter 5.3 some actual examples are presented instead, in order to show the possibilities and methods used.

Bekaert has gained comprehensive experience in choosing the suitable design approach per application and has a broad overview of the existing design regulations per country.
5.3 Examples

The following examples represent the majority of steel fibre concrete applications today, which are subject to bending with or without compressive forces. Design approaches or other guidelines to be applied may depend on the actual location of a project. Thus, no detailed description of the design methodology is given for the different examples.

Bekaert will assist you in selecting and applying suitable methodology.

5.3.1 Strip foundations and foundation beams

Strip foundations are commonly used to support line and point loads resulting from load bearing walls or columns. Depending on the actual load configuration, element dimensions have to be defined accordingly.

The most common type of strip foundation is loaded by somewhat constant (line-) loading, resulting in a rather steady soil pressure, see Figure 27 at top. The dimensions of such strip foundations may be chosen based on the internal angle of stress distribution in unreinforced concrete. However, most designers prefer to have at least some minimum ductility to allow for shrinkage and unforeseen movements. Steel fibres are often used to replace this minimum reinforcement, based on minimum performance criteria according to national approvals such as [64], [65] or international standards [1]. Higher ductility may be provided in certain situations.

Figure 27: outline of ground pressure distribution for strip foundation (top) and foundation beam (bottom)
In the case of non-uniform loading or significant point-loads, but also for wide strip foundations, significant bending moments have to be taken into account. Such strip foundations as indicated in Figure 27 (bottom) need to be designed as a beam on ground, in the longitudinal direction and/or in the transverse direction. Calculated bending moments and thus required performance of steel fibre concrete depends on parameters like soil bearing capacity, beam geometry and load pattern. Existing design rules may be used to determine fibre concrete performance. In Belgium, for instance, [66] covers this application.

5.3.2 Industrial floors and pavements

Steel Fibre reinforced industrial floors and pavements have become the state of the art in many countries around the world. Relevant design methods assume specific models for the interaction between floor slab and sub-base. They differ principally by the degree of simplification that is taken into account.

The major benefit in using steel fibres is very obvious here: no reinforcement cage needs to be prepared and placed in a trench while the whole project is still in the ground. Low bond quality between reinforcement and concrete matrix due to mud-covered steel surface does not occur. Required time for foundation works is reduced to a minimum, putting the project on a dry and clean basis as quickly as possible.
Usually the determination of section forces for industrial floors follows the Westergaard method [67], which is based on the elasticity theory. For steel fibre concrete, however, yield-line theory has turned out to be a more accurate and thus more economic design approach [68]. Reason for that can, again, be found in the difference between section and system performance as described above, chapter 5.1.2 in particular.

Apart from proper design and accurate detailing, a well compacted sub-base, adequate concrete mix design, good workmanship and careful curing are required. This, by the way, applies for any industrial floor, may it be steel fibre reinforced concrete, plain concrete or reinforced concrete.

Figure 28: detailing example of industrial floor - avoiding contact to fixed or hard points

In most countries, industrial floors are considered as “minor structures”, at least from a structural point of view. Consequently different safety concepts may be adopted compared to “real” structures. However, this may not lead to a poor design approach.

In principle, two different types of floors are distinguished: saw cut floors and “jointless” floors. Saw cuts are introduced at relatively short intervals to enforce cracking. Restrained stresses are thus eliminated or significantly reduced. Typical joint distances are 4 m to 8 m, although others are possible and may even be required.

Spalling of saw cut joints due to heavy and/or frequent traffic loads like forklifts can be prevented by choosing for the so-called “jointless”-floor concept. In this case, no saw cuts are introduced at short intervals but special joint profiles are placed at distances of about 30 m to 40 m. This concept comes along with special design, detailing, concrete composition, placing and curing requirements.
Using steel fibres reduces many disadvantages related to plain concrete industrial floors. Once cracked, plain concrete floors lose important serviceability properties and even load bearing capacity may become critical, at least in the mid term. Steel fibre concrete floors maintain both even in the cracked state, still allowing for higher loadings. Thinner slabs can be executed while leaving load-bearing capacity unchanged. In addition, impact resistance is improved whereas crack resistance is increased (chapter 2.8).

Compared to reinforced concrete, no mesh needs to be installed so that concrete trucks can directly drive by and pour the concrete right in place (Picture 13, left). There is no need to worry if the reinforcement was placed in the right position. It allows for quicker construction and does not hinder the use of sophisticated methods like laser-screed (Picture 13, right) and topping-spreaders, which do improve speed and quality.

These and other benefits have convinced innumerable customers around the world, may they be local producers or multi-national companies.
Bekaert provides detailed technical support concerning designing, detailing and execution of steel fibre concrete industrial floors. An online-version of Bekaert’s design software Dramix® Pro and technical reports explaining the design methodology are available.

5.3.3 Floors on Piles

In some countries, for example The Netherlands, United Kingdom and Sweden, actual ground conditions require to found industrial floors on piles. The alternative of a stable and well-compacted sub-base is either not available or would raise costs to unacceptable heights.

Three different approaches are considered today; see Figure 30:

a) steel fibres without any additional reinforcement
b) steel fibres in combination with reinforcement along the pile grid
c) steel fibres in combination with a mesh above the piles

Despite a “fibre only” solution a) being most favourable from a practical point of view, Figure 23 shows that for this application a combined reinforcement performs better with greater ductility. Even if high performing steel “fibre only” concrete was used, 40 kg Dramix® RC-80/60-BN in the related test, crack widths cannot always be limited in terms of serviceability, once the yield lines have formed. Placing some reinforcement at the right position changes this significantly, b) and c). Intrinsic safety increases while the required performance of steel fibre concrete reduces, providing that the comparison is made on the same basis.

![Figure 30: outline of floor on pile approaches](image-url)
When steel fibre reinforced floors on piles were introduced into the market, grid reinforcement b) was dominant (Picture 14). Design was based on large-scale testing and related recommendations [69] in combination with [33].

![Picture 14: grid reinforced floor on piles](image)

In the meantime, the solution with a top mesh above the pile c) has become more and more popular due to easier and quicker placing of reinforcement (Picture 15). Recently, an official design procedure for this methodology was published in the Netherlands [84].

![Picture 15: pile mesh reinforced floor on piles](image)

As with traditional industrial floors, Bekaert provides detailed background information for the design of a floor on piles [70].
5.4.3 Under water concrete slab

Quite often, building pits have to be excavated below the water table. To get a dry temporary work space, underwater concrete slabs are combined with circumferential (sheet-)pile walls. Once the slab has hardened, the water inside is pumped out. Wall and slab may now be seen as a vessel that wants to float in the surrounding water. In order to prevent this, the slab has to be tied down by means of ground anchors. The difference in water pressure $\Delta h$ results in significant loading on the slab, see Figure 31. Turned through 180°, the slab may be interpreted as a floor on piles as explained in chapter 5.3.3.

![Figure 31: principle of underwater-concrete slab, pressure difference $\Delta h$](image)

Casting concrete under water requires not only special provisions for the concrete mix-design but also the use of divers. Otherwise, concrete placing cannot be monitored and controlled. Installing reinforcement is either not possible at all or extremely expensive in time and money. This is why underwater concrete is typically plain concrete. However, the disadvantages of using a brittle material such as plain concrete are thick slabs, narrow anchor distances and the potential risk of sudden failure.

Used for the first time at the Potsdamer Platz in Berlin, Germany, steel fibre concrete has turned out to be a suitable alternative. Providing sufficient post crack strength to the concrete, the system maintains load-bearing capacity even in the cracked state, see Figure 23. As it was the first use and due to the actual size of 20,000 m² and a water pressure of 180 kN/m², laboratory and real scale tests were made to ensure load bearing capacity and workability.

![Picture 16: building pit at Potsdamer Platz after pumping out](image)
As underwater concrete slabs are temporary constructions and not subjected to dynamic loading (contrary to floor on piles), there are no objections to using steel fibres as the only reinforcement if existing experience is followed. Design should be carried out according to yield-line theory [71], but even linear-elastic design can deliver economic results in some cases.

So far, the vast majority of steel fibre concretes used for this application had a post crack flexural strength just below or similar to the uncracked flexural strength. Using 40 kg/m³ Dramix® RC-80/50-BN with a fibre network of about 17,000 m per m³ has been the first choice at Potsdamer Platz and a variety of other projects.

Major savings can be achieved by reducing slab thickness and, as the consequence, excavated volume and wall height, also by increasing anchor distance and thus reducing the number of anchors. In addition, a ductile failure mode is achieved. Plain concrete would fail immediately.

5.3.5 Rafts and foundation slabs in housing

Using elastically supported steel fibre reinforced concrete slabs as foundations slabs appears to be the natural evolution of an industrial floor. Experience with foundation slabs for the housing market exists in countries like the Netherlands, Germany, Austria and Switzerland – in some cases even since the mid of the 1990s. Design guidelines [20], [26] or type approvals [22], [31] are used for designing and detailing. Certain requirements concerning soil quality, maximum dimensions, environmental conditions etc. have to be taken into account.
In many of the cases, traditional reinforcement can be completely replaced by steel fibres. Major time savings can be achieved and error rate reduces. Incorrectly placed reinforcement or insufficient concrete cover can not occur as a matter of fact.

Due to the nature of the application, performance certified steel fibre concrete is essential for such an application, see chapters 2.6 and 4. Rafts for higher loads than those typically associated with housing applications are often built using combined reinforcement. Examples may be found in chapter 7.3.

Bekaert has prepared tools, approvals and background information for designing load bearing foundation slabs.
5.3.6 Wall

Steel fibre reinforced walls have a long tradition in housing applications.

Typically, steel fibre reinforced concrete walls are calculated as single span beams with or without a clamp support at bottom or top. Design does not vary a lot from traditional reinforced concrete wall design.

As with load bearing foundation slabs, design guidelines [20], [26] or type approvals [23], [72] are available for cellar walls or walls subject to other bending moments. With steel fibre concrete, there is no reinforcement to delay the erection of the formwork or hinder pouring.

Only in very simple cases, wall design may be based on plain concrete design. Steel fibres would then still be justified as temperature and shrinkage reinforcement. Buckling, however, needs to be excluded by choosing an appropriate wall thickness.
Combined reinforcement may be used if increased serviceability requirements have to be met or loading would exceed the load bearing capacity of steel fibre concrete, see chapter 7. In the case of significant uplift forces, additional reinforcement will be required. Compared to a traditional reinforced concrete wall, reinforcement layout can be greatly simplified.

Bekaert has prepared tools, approvals and background information for easy designing of cellar walls.

5.3.7 Pipes

Steel fibre reinforced concrete pipes have been available for a long time. However, in recent years there has been a very significant increase in their use. This may be related to the publication of the European standard EN 1916 in 2002 [73], which specifies performance requirements and describes test methods for precast concrete pipes and fittings used for the conveyance of sewage, rainwater and surface water. Design is based on full scale testing.

Asking for more severe requirements than for those made from reinforced concrete [73], the higher quality of steel fibre reinforced pipes became obvious – and convinced a significant number of pipe producers around the world.
For steel fibre concrete pipes, the test procedure consists of 5 phases, as explained below and indicated in Figure 33.

- **Phase 1-2:**
  The pipe has to withstand a proof load of 0.67\( F_n \) for one minute without showing any crack. \( F_n \) is defined as the minimum crushing load. It is depending on both pipe nominal size and strength class.

- **Phase 3:**
  If no crack is found the load is increased to the ultimate (collapse) load \( F_u \) which has to be higher than the required minimum crushing load \( F_n \).

- **Phase 4-5:**
  Once the load has fallen to 95\% or less of the ultimate load \( F_u \) it is fully released. Then 0.67\( F_n \) is reapplied for one more minute without the pipe collapsing.

**Figure 33: crushing test for steel fibre pipes according [73]**

In the prescribed crushing test [73], reinforced concrete pipes are just required to show a stabilized surface crack, with a width not exceeding 0.3 mm over a continuous length of 300 mm. Steel fibre reinforced concrete pipes, on the other hand, have to withstand a proof load of 0.67 \( F_n \) without showing any crack in the first phase of the crushing test. Furthermore, reinforced pipes do not need to comply with the crushing test requirements of phases 4 and 5.

The difference in allowed cracking and crack width at the different stages of the test program also means that steel fibre pipes will meet higher water-tightness requirements. Due to fibres being distributed throughout the whole geometry of the pipe, seals and other critical parts are much better protected against handling damage. The number of scrap-pipes can be reduced enormously.

Bekaert provides special documentation on request.
5.3.8 Segmental Lining

The use of tunnel boring machines (TBM’s) and thus the installation of segmental linings has become more and more common in the tunnelling world. Particularly in urban areas or geologically difficult situations, the use of a TBM proves to be very attractive option as it provides continuous support, even with small overburden depths. The tunnel opening is lined immediately behind the TBM using mostly precast concrete segments. This of course reduces the risk of settlement for buildings in the neighbourhood of the tunnel.

![Picture 23: Installation of a segment in a tunnel lining](image)

The segments are precast at the job site or are brought in from a remote specialized precast concrete plant. The lining consists of concrete rings, which are formed by putting several single pieces together. In most ground conditions, the segments only have to resist normal compressive forces in their final position.

Design can be done by calculating the m-n-equilibrium for the tunnel lining section forces according to chapter 5.2. Actual bending moment capacity “m” is influenced by the occurring axial forces “n”. Very often, the m-n-envelop is used to compare maximum load-bearing capacity to relevant combinations of bending moment and axial force (Figure 34).

![Figure 34: example of m-n-envelope](image)
However, these segments are subject to different loading conditions before they get into their final position. The precast segments have to resist bending moments and flexural stresses when being demoulded and transported to the storage facilities located outside the precast building. They have to resist tensile thermal stresses due to temperature changes at the storage area. The heaviest loading, however, takes place when the segments are being installed and they have to resist the very high jack or ram loads as the TBM moves forward.

Cracking and spalling are the main problems of reinforced concrete segments. The heavy jack or ram loads are applied at the outer unreinforced concrete skin of the segments (Picture 24). When spalling occurs, a traditionally reinforced segment has to be repaired or even replaced for obvious reasons of durability concern. This, however, delays the tunnel construction progress and is a very expensive operation.

![Picture 24: hydraulic jacks induce complex stresses](image)

The very complex tensile stress pattern induced by these loads requires a quite complicated steel reinforcing cage, which makes it heavy and expensive. Steel fibres are a technically viable alternative to the traditional mesh and rebar reinforcement. The homogeneous fibre distribution makes it possible for the concrete segment to absorb tensile stresses at any point and in any direction. Cracking and spalling resistance are considerably improved, resulting in lower damage and less rejected elements [74], [75].

Designing these stresses has turned out to be quite difficult, both for reinforced concrete and steel fibre concrete. Design by testing or using existing experience are often considered as the preferred alternatives to a theoretical design.

Various reference projects have proven that tunnels with a diameter of up to 7 – 8 m, and recently at Önzberg in Switzerland with a diameter of even more than 10 m, can be lined with steel fibre reinforced concrete segments without the need for any other additional reinforcement.
For large diameters and/or in difficult ground conditions it may be recommended to consider using a combined reinforcement of rebar and steel fibres. In any case, steel fibre reinforced segments offer new possibilities, both in technical and economical terms.

Bekaert has published specialized and detailed information about the art of using steel fibres in tunnelling, also covering shotcrete and mining applications [34], [76].

6 SFRC in Shear

Using steel fibres to enhance shear capacity is a relatively new field of application, at least when compared to bending moment capacity. Nevertheless, the effect of steel fibres is even more obvious here. Avoiding brittle failure requires much less steel fibre concrete performance for shear than is the case for bending.

Shear failure, however, is generally not associated with those steel fibre structures which are common today and which typically do not have additional steel reinforcement or major axial forces. In most cases, the governing failure mode is bending, as is the case for industrial floors, foundation slabs and other relatively thin structures with dominant bending. Most design recommendations for shear require bending reinforcement, confirming the principle of dominant flexural failure for steel fibre concrete without additional reinforcement.

Whenever traditional steel reinforcement is used or actual project conditions are beyond the existing experience for “fibre only” applications, shear capacity needs to be verified.
6.1 Section based approach

Unlike bending, shear design is always a section based design approach. Consideration of system properties is not necessary. Experience from testing shows that steel fibres can replace the minimum shear reinforcement which is to be placed for robustness reasons when no, or almost no, shear reinforcement would be required due to the level of the actual forces.

For higher shear forces, additional shear reinforcement such as stirrups need to be placed. The fibres, however, will allow a reduction in their spacing and/or diameter. The most economical applications are those where all shear reinforcement can be replaced by steel fibres.

6.2 Shear

Adding steel fibres to a reinforced concrete member without shear reinforcement will allow for a ductile failure mode as would be the case when using stirrups. Important research, for instance, has been carried out in the course of the Brite-Euram project and was published accordingly [41].

Figure 35 compares a reinforced concrete member (bending reinforcement) without any shear reinforcement to one with steel fibres as the only shear reinforcement.

![Figure 35: reinforced concrete beam without and with steel fibre shear reinforcement](image)

The shape of the shear crack is significantly different when steel fibres are used. Instead of quick crack propagation in the member without shear reinforcement (red line in Figure 35), crack propagation becomes stable and controlled over a wide range (blue line in Figure 35) with steel fibre reinforcement. Ductile failure is achieved where no shear reinforcement results in sudden collapse.

The effect of steel fibres on shear capacity may be explained due to the fibres dowelling the cracked section thereby maintaining aggregate interlock and providing an additional vertical component, supporting the internal equilibrium of forces.
Actual design recommendations suggest steel fibres as shear reinforcement in reinforced concrete beams, e.g. [28], [27], [30], [41] or [77]. Even pre-stressed concrete beams at spans up to 25 m or more have been reported [78].

The required post-crack strength (chapter 2.4) depends on several conditions and is also dependant on the actual design recommendation to be applied for the reinforced concrete itself. Therefore, no design details shall be explained here. The suitability of the chosen design approach, practical conditions and existing quality control should be verified in any case.

Bekaert is provided to support you in defining the right steps.

6.3 Punching shear

Punching resistance is also increased by adding steel fibres to concrete. The above explained mechanisms and conditions are also applicable for this application.

For steel fibre reinforced concrete slabs on ground, still the most important application of steel fibre concrete, punching is usually not seen as a critical design case. Remember, bending is the governing load case in these applications. Due to that lack of need, limited information is available on the punching of slabs with steel fibres as the only reinforcement, both for shear and bending.

Recent testing [79] has confirmed that bending may be assumed as the governing failure mode in steel fibre concrete slabs. By gradually increasing the amount of traditional steel reinforcement, the failure mode gradually shifted from bending to punching. For a reinforcement ratio lower than about 0.1%, the bending moment capacity, not the punching shear capacity, was governing. A plain concrete specimen failed brittle, as expected, but both steel fibre and combined reinforced specimens failed ductile.

This may explain why most of the testing has been carried out on reinforced concrete containing steel fibres as punching reinforcement. Taking [80] as a special example, steel fibres, under certain conditions, can even outperform headed shear studs as punching reinforcement. In the mentioned investigation, punching shear strength and deformation capacity of concrete slab-column connections was investigated under earthquake-type loading. Figure 36 shows that 1.5% by volume Dramix® ZP305 steel fibres can enhance ductility and robustness enormously while maintaining the shear strength of the slab-column connection. The fibres only need be used within the area critical to punching – which would require a high level of quality control in a real project. The specimen reinforced with shear studs as punching reinforcement did not contain fibres. Failure occurred much earlier in this case.

The longitudinal reinforcement was the same in both cases.
Figure 36: hysteresis envelope of lateral load vs. drift in a columns-slab connection under earthquake-type loading, based on [80]

Of course, this is an extreme example, chosen to demonstrate the general potential of steel fibres in punching. However, even in normal design conditions steel fibres can offer alternatives to increase punching shear strength.

As already explained in chapter 6.2, no design details shall be explained here. The suitability of the chosen design approach, practical conditions and existing quality control should be verified in any case.

Again, Bekaert is able to support you in defining the right steps.

6.4 Examples

The following examples have been chosen to give an overview of the use of steel fibres as shear reinforcement. All examples are prefabricated elements. For the time being, existing experience with shear is restricted to such applications.

The range from simple lintels, large span pre-stressed beams and highly loaded coupling elements shows the possibilities. Engineers can imagine which other applications could benefit from steel fibres as shear reinforcement, either alone or in addition to conventional shear reinforcement.

6.4.1 Prefabricated Lintels

Prefabricated lintels are commonly used in residential or industrial buildings. Especially in masonry walls, prefabricated concrete lintels are an important element in order to bridge openings quickly and easily.
Despite their small dimensions, many lintels require both bending and shear reinforcement. One can imagine that making and placing stirrups for small cross sections is a difficult and expensive task. An alternative is the combination of longitudinal reinforcement with steel fibres as shear reinforcement. Depending on the actual shear stress level, all stirrups may be replaced by steel fibre concrete.

Figure 37: reinforcement layout of traditional lintel and lintel with steel fibres as shear reinforcement

Major achievements in production time and costs may be expected, especially if the required length of a lintel can be sawn from a continuously produced "lintel-band".

Picture 26: example of lintel with steel fibre shear reinforcement

Those lintels can be designed but design by testing is often the preferred choice in this specific case. Typical dimensions are found in the following range:

- length: ~ 90 cm – 320 cm
- width: ~ 11.5 cm – 24 cm
- height: 14 cm – 24 cm

Picture 26 shows such a lintel in an intermediate construction stage. In this case, the lintel would contain 15 kg/m³ to 30 kg/m³ Dramix® RC-65/35-BN, depending on the actual loading. Bending reinforcement is contained additionally.
6.4.2 Pre-Stressed Beams and Roof decks

In [78] practical experience with pre-stressed beams made from self-compacting steel fibre concrete is described. All traditional reinforcement was replaced by either pre-stressing tendons or steel fibres. Large scale testing proved the benefits and safety of this construction method.

Figure 38 shows the load versus mid-span deformation curve of such a beam. Despite the first shear cracks appearing at a stress only about 50% above the designed service level, the steel fibres effectively prevented brittle failure. The beam actually failed after exceeding more than three times its calculated service load. Multiple shear cracking could be observed, according to [78].

![Load vs. mid-span deformation](image)

**Figure 38: load vs. mid span deformation of a pre-stressed beam with steel fibres as shear reinforcement according [78]**

The same principle was applied in an application for pre-stressed roof decks. The curved, V-shaped cross section (Figure 39) of these elements made the incorporation of stirrups and web reinforcement very difficult and costly.

![V-shaped cross section of roof-deck](image)

**Figure 39: outline of V-shaped cross section of roof-deck**

Dramix® RC-45/30-CN, a zinc coated steel fibre of 30 mm length, was dosed at 50 kg/m³ in order to meet the required shear strength so that all stirrups and web reinforcement could be replaced.
Picture 27 shows an example of these elements.

![Picture 27: pre-stressed roof decks with steel fibers as shear reinforcement](image)

6.4.3 Coupling Beams

In high-rise buildings, coupling beams are a common method of connecting two shear walls. The building becomes stiffer while maintaining ductility at the same time. In a seismic event, the coupling beams act as plastic hinges and absorb part of the energy. Preferably, the hinges should localize in the beams rather than in the shear walls to allow for later repair.

Translated into practice, this means a very heavy and very complicated arrangement for the reinforcement (Picture 28). The time needed for installation is often critical to the progress of the whole building.

![Picture 28: traditionally reinforced coupling beam](image)
Using an engineered steel fibre reinforced concrete allows simplifying the reinforcement layout enormously. Even prefabricated elements become possible. However, due to the acting forces and due to the required ductility, only specimens containing 120 kg/m³ Dramix® RC-80/30-BP have proven to work so far. The plastic hinges now form in the coupling beams and damage to the shear walls is limited.

Despite using high performing high strength steel fibres at such high dosages, the application provides economic advantages due to much easier and quicker building progress – in addition to the unmatched technical advantages, see Picture 29.

7 Combined Reinforcement: 1 + 1 = 3

In recent years, the combination of steel fibres and reinforced concrete has become more and more common, at least in those countries where broad experience with steel fibre concrete already exists. The benefits of each of the two materials can be used together in order to improve load bearing capacity, serviceability and durability of a structure while still simplifying construction and thus reducing required installation time. Combined reinforcement is often seen as one possibility to answer today’s requirements on structures in regards to increased durability requirements, enhanced life time and reduced total costs of ownership.

Sometimes, 1 plus one does not equal 2 - but 3.
7.1 Load bearing capacity

Adding steel fibres to reinforced concrete will increase its load-bearing capacity – and vice versa. The additional effect of reinforcing steel can be taken into account when calculating the M-N-equilibrium of a section. Based on a stress-strain relation for steel fibre concrete, the stress-strain relation for traditional reinforcement may be incorporated. This requires that there is the right correlation between steel strain and the fictitious steel fibre concrete strain. Very often, maximum steel fibre concrete strain is equated to maximum steel strain.

Figure 40 shows load-deflection curves for reinforced concrete beams. One beam is reinforced with eight re-bars, diameter 6 mm and one beam is reinforced with five re-bars, diameter 6 mm. The amount of steel fibres in the third beam was determined to replace three re-bars diameter 6 mm. The required calculations were carried out in accordance with [27], based on residual post crack strength derived from standard beams according to [27]. The maximum strain of both traditional reinforcement and steel fibre concrete was taken as 2.5%.

![Load deflection curves of reinforced concrete beams with and without hooked-end steel fibres](image)

The two components, steel fibres and reinforcing bars, complement each other, thus increasing the load-bearing capacity of the combined section (Figure 41).

![Stress-strain relation of a) steel fibre concrete and b) combined reinforcement](image)
Recent design approaches for steel fibre reinforced concrete cover the design of combined reinforcement. As for reinforced concrete, the equilibrium of internal forces is calculated on the basis of strain iteration for the combined reinforcement. Spreadsheet calculation is a quick and simple means to perform that iteration.

As a first approximation and for a preliminary estimate of combined bending moment capacity, the bending moment capacity of the steel fibre concrete may be added to that of the reinforced concrete, see Figure 42. If axial forces are present, this simplified approach should not be applied.

$$d \approx 0.9 \cdot d \cdot A_s \cdot f_y$$

$$f_{fl,II} \cdot Z$$

Figure 42: approximation of bending moment capacity in combined section (d = static height, $A_s$ = steel section, $f_y$ = steel strength, $f_{fl,II}$ = post crack flexural strength and $Z$ = section modulus)

However, an accurate calculation of the section equilibrium should always be performed according to the applied design rule and as outlined in Figure 41.

### 7.2 Crack Width

Due to the nature of the material, cracks in reinforced concrete are inevitable. If these cracks do not exceed a certain width they are neither harmful to a structure nor to its serviceability. Design standards for reinforced concrete such as [33] point out durability and serviceability aspects. This immediately leads to the limitation of crack widths. The width of these cracks mainly depends on concrete tensile strength and the amount of steel provided, but also on the type of loading, thickness, concrete cover and reinforcement diameter. As a simplification, crack width $w_{RC}$ of reinforced concrete may be seen as a function of concrete tensile strength $f_{ct,i}$.

$$w_{RC} = \text{function}\{f_{ct,i}\}$$

**Equation 4**

Using fibres in combination with traditional reinforcement will change neither concrete tensile strength nor type of loading. Nevertheless, as some part of the load will be carried over the crack by the steel fibres, only part of the energy is released compared to concrete without steel fibres. Consequently, required anchorage length of the traditional rebar reduces and thus crack distance decreases with the number of cracks increasing. As a result, crack width reduces for a given deformation.
Assuming a concrete tensile strength of 3.0 MPa, for example, while providing a post crack tensile strength of 1.0 MPa when using steel fibres, only 2/3 of the full crack load has to be considered for the design of crack control reinforcement. This has a strong effect on the required steel section. Savings of up to 50% are possible when using high-performing steel fibres.

Furthermore, steel fibres also reinforce the concrete cover - which is definitely not the case for reinforced concrete alone. In addition, the whole section benefits from the fibres, even outside the effective zone within which the reinforcement determines the crack width.

A number of related test programs have been carried out and a number of design methods have been proposed, [27], [41], [57] and [85]. These approaches more or less follow the same principle differing only on a few points. This appears to be due to the underlying design approach for the reinforced concrete itself. But all of them have one thing in common, namely the reduction of the concrete tensile strength \( f_{ct,i} \) by post crack tensile strength \( f_{ct,ii} \).

In the case of combined reinforcement, Equation 4 may now be adjusted to be a function of first crack tensile strength \( f_{ct,i} \) minus post crack tensile strength \( f_{ct,ii} \).

\[
\text{w}_{\text{comb}} = \text{function} \left\{ f_{ct,i} - f_{ct,ii} \right\}
\]

Equation 5

It is important to use the two strength values applicable at the same concrete age. In the case of dissipation of hydration heat, for instance, both \( f_{ct,i} \) and \( f_{ct,ii} \) have to be reduced by the same percentage, approximately 50% according to [33]. Time dependency may be eliminated by proceeding as described in Equation 6.

\[
\text{w}_{\text{comb}} = \text{function} \left\{ f_{ct,i,ef} \cdot (1 - f_{ct,ii,28} / f_{ct,i,28}) \right\}
\]

Equation 6

where:
- \( f_{ct,i,ef} \) concrete tensile strength effective at cracking
- \( f_{ct,i,28} \) first crack concrete tensile strength at 28 days
- \( f_{ct,ii,28} \) post crack concrete tensile strength at 28 days

Using combined reinforcement has a very positive effect on both quality and speed of construction. The influence on quality may be seen from different perspectives. As a rule of thumb, the required amount of reinforcement can be reduced up to 50% when using high performing steel wire fibres. This allows for larger distances between re-bar and/or smaller diameters while keeping crack widths at the same level. At the same time the installation process will become less complicated, more accurate and faster. Pouring and compacting concrete becomes easier and better results are likely. In many cases, rebar can be replaced by welded mesh so that even more time is saved and different building methods can be applied. The latter can contribute to significant improvements in the whole construction process and the overall quality. Or, as another option, quality can be improved by simply adding steel fibres in addition to the foreseen reinforcement.
Very often short construction times and high quality conflict on a construction site. In the case of combined reinforcement, durability, serviceability and construction time may be improved in one pass. Significant savings of maintenance costs may also be achieved. Therefore, it is advisable to evaluate costs and savings for structures with combined reinforcement over their full life time.

Examples are given in chapter 7.3.

7.3 Examples

A variety of projects has been built with combined reinforcement in countries all over the world. A few examples, most of them published in [86], shall be used to explain why combined reinforcement was used and what benefits were achieved. Fibres were considered in either the serviceability state alone or in both the serviceability and ultimate limit states.

7.3.1 Repair of an industrial floor

Located in an industrial park in Ismaning (Germany), the surface of an existing saw-cut floor was damaged after some years in use. A layer of 8 cm was milled and replaced by a new 8cm floor with combined reinforcement. A Q295 mesh (steel section 295 mm²/m in both directions) was combined with 35 kg/m³ Dramix® RC-65/60-BN (end-hooked, l = 60 mm, d = 0,9 mm).

The new layer was separated from the old slab by a double layer of plastic sheet. Despite the thin concrete layer, larger pour sizes were achieved compared to the usually accepted sizes for 8 cm overlays. Saw-cuts were avoided so that flaking edges were not inevitable. The new joint layout, using special joint profiles with edge protection, could be aligned with the current needs and did not need to match with the old saw-cuts. The joint layout was 27 m x 30 m, total size 3,400 m².
Construction was simplified and sped up due to the use of a light mesh. A concrete pump was not needed as the truck mixers could drive directly to the pouring point. Laying the light mesh was carried out as the concrete was placed. The calculated crack width of 0.2 mm was not exceeded.

### 7.3.2 Industrial floor as secondary barrier

An industrial floor had to be designed as a secondary barrier against hazardous substances eventually leaking from their containers. The production facility is located at Waldenburg, Germany. Calculated crack width needed to be limited to 0.1 mm, following the regulations in [38]. A 20 cm thick slab was poured on top of a blinding concrete which had been power floated. A double layer of plastic sheets separated the two layers in order to reduce stresses due to restraint deformation. Joint distances approximately about 30 m x 30 m. A top Q513 mesh (steel section 513 mm²/m in both directions) was combined with 30 kg/m³ Dramix® RC-80/60-BN (end-hooked, l = 60 mm, d = 0.75 mm). The total size of the project was 15,000 m².

Instead of using a double layer of reinforcement consisting of a large number of single re-bars, a strong mesh was combined with high performing steel fibres. The required time for installing reinforcement was significantly reduced. The mesh placing was undertaken simultaneously with concrete pour and a concrete pump was not required. As a consequence of these changes, using a laser-screed and topping-spreader became a new option, not previously considered.

A laser-screed compacts and levels the concrete automatically and simultaneously at high speed. With a topping-spreader, the correct amount of dry-shake material can be applied "dry in fresh". Good surface quality and abrasion resistance after power floating and finishing could then be achieved.

![Picture 31: laser-screed compacting and levelling concrete](image-url)
This would not have been possible if two layers of mesh or one top layer of rebar had to be placed. Pumping the concrete would have been inevitable, installing reinforcement and pouring the concrete would have been two consecutive steps. Manual compaction and levelling could not have been done at the same time and nor would a comparable level of quality have been achieved compared to using a laser-screed and topping spreader.

7.3.3 Load-bearing joint free industrial floor

The industrial floor of a handling facility for metal scrap in St. Gallen (Switzerland) was foreseen as a joint free slab with dimensions of 100 m x 40 m. Loads originated from metal scrap and the steel structure of the building. Retaining walls, clamped to the slab, allowed heaping of scrap up to more than 5 m. Heavy impact loads from scrap handling also had to be considered.

As the slab was cast under exterior conditions before the actual building was finished, restraint deformations due to temperature were also likely to occur.

The slab was fully restrained by connecting it to the strip foundations of the exterior walls. Therefore, crack-inducing deformations were likely to occur and quite quickly. For durability reasons, a calculated crack width of 0.2 mm was required.

Looking at the total cost of ownership, the investor preferred combined reinforcement to reinforced concrete. Finally a 25 cm slab was cast with a C30/37 concrete containing 30 kg/m³ Dramix® RC-65/60-BN (end-hooked, l = 60 mm, d = 0.9 mm) and reinforced with two layers of rebar Ø12@150 mm. In addition to the positive effect on crack width, the fibres also provided resistance to impact, originating from the scrap handling. A more durable floor was expected compared to the reinforced concrete option.
7.3.4 Raft and Walls in SLS

Foundation slab and cellar walls of an office building for the local tax authorities in Hersbruck (Germany) had to be designed for a calculated crack width of 0.1 mm. This severe requirement was due to the ground water pressure. A C25/30 concrete was used containing 30 kg/m³ Dramix® RC-80/60-BN (end-hooked, $l = 60$ mm, $d = 0.75$ mm).

The 60 cm thick slab, ~43 m long and ~11 m wide, had to be reinforcement with $Ø14$ mm rebar at top and bottom. An alternating bar spacing of 35 mm and 100 mm respectively was applied to allow for sufficient workspace. For the 25 cm walls, a $Ø10@150$ mm mesh plus additional $Ø10@200$ mm rebar to both the in and outside faces was sufficient.

![Picture 33: Raft with alternating distance of reinforcement](image)

The amount of reinforcement needed was still high. However, compared to reinforced concrete, smaller diameters were used at a spacing that still enabled the workers to place and compact the concrete correctly. This, of course, is almost as important as an adequate amount of reinforcement if a crack requirement of 0.1 mm is to be met in reality.

7.3.5 Raft in SLS and ULS

At Södertörn’s court house in Flemmingsberg (Sweden) combined reinforcement was used to reinforce the load bearing foundation slab.

The 60 cm thick slab had an uneven bottom and was founded on rock, crushed rock and piles. Water pressure of 15 kN/m² had to be compensated by the dead weight of the slab to avoid floating. Required crack control was provided with a top layer of $Ø10@100$ mm reinforcing bars and 40 kg/m³ Dramix® RC-80/60-BN (end-hooked, $l = 60$ mm, $d = 0.75$ mm).

As the effect of fibres was also taken into account for ultimate limit state, additional reinforcement was only required locally, especially at the bottom where load-bearing columns of the building were located. Slab dimensions are ~ 32 m x 30 m. Pumping and placing concrete did not cause any problems, very well supported by a suitable concrete composition. Reducing and simplifying the reinforcement and thus reducing construction time led to major savings for the contractor.
Picture 34: Raft founded on rock, crushed rock and piles

Calculated crack width was 0.2 mm. Despite the maximum dimensions of 32 m and being connected to the ground, no cracks, which exceeded the design limits, could be found even half a year after execution.

7.3.6 Columns in high-rise buildings

The new 276 m high tower of CCTV (China Central Television), in Beijing (China) is a very impressive example of today's architectural and engineering ideas. Positioned at the diagonal corners of a rectangular base, the two towers had to be built with an inclination. This was meant to compensate for the deformation arising from the top cantilevering floors, which projected towards a third corner of the rectangular basis. During the construction phase, before the two cantilevers had been connected, significant deformation of the main load-bearing columns was expected. Thus, very heavy reinforcement had to be used, comprising thick reinforcing bars at close centres.

Picture 35 a/b: column reinforcement detail and CCTV tower (modell)
Apart from deflections, additional stresses were expected due to the concrete shrinking against the heavy reinforcement. 25 kg/m³ of Dramix® RC-65/35-BN (end-hooked, l = 35 mm, d = 0.55 mm) was added to allow for better crack control, based on sound engineering judgment. A self-compacting concrete was used, meeting the engineer’s requirements and expectations.

The Olympic games of 2008 were broadcast from this building.

Due to the limited knowledge of the effect of steel fibre concrete on buckling, combined reinforcement (steel fibres in addition to longitudinal bars and stirrups) is always recommended for columns in such a critical design case.

### 7.3.7 Concrete face of a rock-filled dam

Initially, reinforced concrete was foreseen for the concrete face of the rock-filled dam of the Longzhou 2nd phase project in China.

The highest point of the dam is 146.5 m. It is located in an area with frequent seismic activity, dry and cold weather with large temperature differences from day to night. Following this, the main reinforcement ratio was designed as 0.4%, the transverse reinforcement ratio as 0.35%. However, some of the reinforced concrete panels showed unacceptable cracking during construction. Most likely, the restraint deformations, arising from the concrete face shrinking against the dam, were exceeding the calculated values. Quite some repair efforts had to be undertaken from start-up, and consequently meeting the required durability of the concrete face was still questioned (dark-grey contours below the red line in Picture 36).

![Picture 36: concrete face of rock-filled dam, with traditional (below red line) and combined reinforcement (above red line)](image)

Therefore it was decided to add 35 kg/m³ Dramix® RC-80/60-BN (end-hooked, l = 60 mm, d = 0.75 mm) to the concrete mix, leaving the reinforcement ratio unchanged. The longest steel fibre panel had a length of 75 m. Panel thickness varied from 70 cm at the dam base to 30 cm at the crest. Never during construction or even after two seismic events did actual crack widths exceed the accepted values for the combined reinforcement.
7.3.8 Oceanographic Park

Combined reinforcement was used for a thin shell structure in the Oceanographic Park of Valencia, Spain. Due to the curvature and the very limited shell thickness of 6 cm to 12 cm, it would have been very difficult to install complicated traditional reinforcement in an accurate and safe way. A maximum diameter of 8mm rebar was preferred due to required concrete cover and the need to curve the reinforcement, especially in the “valleys” of the shell. As the reinforcement had to be placed close to, or even at, the neutral axis of the section, the post crack strength of steel fibre concrete was also taken into account for the ultimate limit state design.

![Shell after completing](image)

Picture 37 a/b: Shell after completing

50 kg/m³ Dramix® RC-80/35-BN (end-hooked, l = 35 mm, d = 0.45 mm) and a single layer of Ø8@150 mm reinforcement were applied, providing durability, serviceability and strength.

8 Outlook

Existing knowledge used in combination with Quality Performance-Concrete (Dramix® QPC), codes and approvals will help to make SFRC even more successful. Reduced construction time, alternative construction methods, simplified reinforcement layout, increased durability at both reduced lifetime costs and environmental impact are some benefits to be considered.

Engineers are empowered to face new challenges by employing steel fibres - but the first step is to get familiar with this construction material. Steel fibre concrete is not to be designed like “ordinary” reinforced concrete. On the contrary, it requires some special knowledge, the right tools and engineering awareness.

Bekaert is equipped to support anyone interested in understanding, designing and applying steel fibre concrete successfully.
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