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# STRUCTURAL APPLICATION OF STEEL FIBRES REINFORCED CONCRETE WITH AND WITHOUT CONVENTIONAL REINFORCEMENT

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## SUMMARY

Structural steel fibre reinforced concrete applications differ mainly from well-known fibre applications like floors and pavements. In structural applications, steel fibres are the main or the secondary reinforcement to take up bending moments and the shear stresses. This paper presents some basic principles governing the structural design of SFRC based on the available design codes. Starting from classification of various types of steel fibres, which have profound effect on the performance of concrete structures, the simple constitutive models are presented, allowing engineer and designer to design with SFRC. Numerous projects have been carried out. A few of them are presented, giving insight information on where steel fibre only reinforcement and combined steel fibre and conventional reinforcement were used.

**KEYWORDS:** steel fibre, bending moment, shear, serviceability, crack, design, concrete.

## INTRODUCTION

Adding fibres in concrete can no longer be considered as new or novel; fibres have been used to reinforce brittle materials, such as straw in mud for masonry construction dating back to the Babylonian and Egyptian eras. From a modern perspective, research into steel fibre reinforced concrete (SFRC) was pioneered by Romualdi and Batson (1963) in the early 1960s where it was demonstrated that tensile strength and crack resistance of concrete can be improved by providing suitably arranged, closed spaced, wire reinforcement. After more than 50 years of research in the development and placement of fibres in reinforced concrete, the concept has matured and its adoption in practice is rapidly developing.

Today, steel fibres are used as main and secondary concrete reinforcement in an increasing number of applications. Well-known and well established applications are, for example, heavy pavements, slab tracks, slab on grade, shotcrete linings and precast applications. More recent SFRC applications can be seen in the domain of structural raft foundations, liquid tight slabs and piled supported slabs, and even bridges and suspended structures.

These developments are steered and boosted by a steady buildup of knowledge as well as the research carried out at various universities and research institutions in order to understand and quantify the SFRC properties. Owing to better knowledge of SFRC, numerous design guidelines, standards and codes have since published; a time line is shown in Figure 1.

New Zealand Standard NZS 3101 (2006) is an early adopter of SFRC in standardization and is largely used the recommendations of the RILEM Technical Committee 162 as reported in Schnütgen and Vandewalle (2003). In the NZ Standard, the post-cracking strength of the SFRC is determined by use of deflection controlled tests on prisms cast with the fibre to be used. This data is then converted to a stress versus crack opening displacement ($\sigma$-COD) relationship using a prescribed methodology. Models for strength and service design in regards to flexure, shear and axial forces are included.

The Australian Standard for the design of Concrete bridges, AS 5100.5 was released on 31st March 2017; this is the first standard in Australia to include procedures for the design of SFRC structural elements.

In Europe a number of national guidelines and technical rules have been established for the design of SFRC structural elements, including the German technical rule for design with SFRC, which have been progressively advanced since 2005; the latest version is the German Committee for Structural Concrete (DafStb) Guidelines for SFRC (2012). Another major source from which design guidance may be found is the International Federation of Structural Concrete (fib) Model Code 2010 (2013), which represents much of the current thinking on the topic from Europe and elsewhere.

This paper presents some basic principles governing the structural design of SFRC based on the current state of practice. Starting from classification of various types of steel fibres, which have profound effect on the performance of concrete structures, the simple constitutive models are presented, allowing engineer and designer to design with SFRC. Numerous projects have been carried out. A few of them are presented, giving insight information on where steel fibre only reinforcement and combined steel fibre and conventional reinforcement were used.

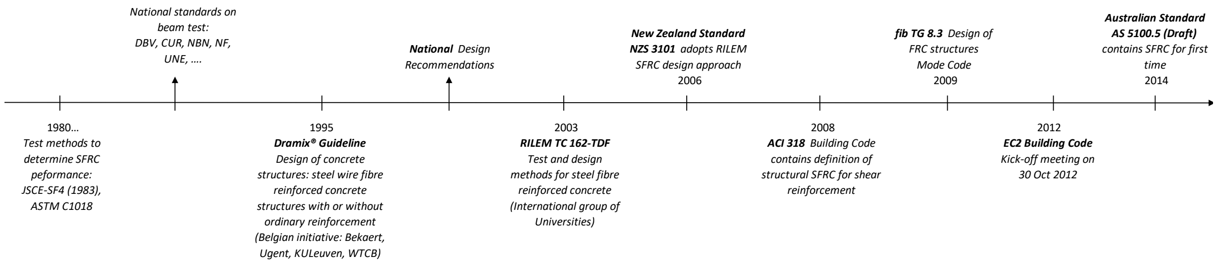



**Figure 1. Evolution of SFRC test and design standards from 1980 to present.**

## MATERIAL BEHAVIOUR OF SFRC

The most important property when designing of a SFRC structural element is its post-cracking, or residual tensile strength. Steel fibres are active as soon as micro-cracks are formed in the concrete. The fibres are able to bridge the crack, transmit stress across the crack and, in the process, provide some resistance to the widening and fracture process of the crack. Thus, unlike plain concrete, an appropriately reinforced SFRC structural element will not completely fail after crack initiation but some residual strength after cracking will be available.

The tensile behaviour of SFRC can either be classified as strain-softening or strain hardening (Figure 2). For strain softening materials, failure occurs over a localised single dominant crack. The behaviour is characterised by the residual tensile strength of the structural element never reaching the tensile strength of the cementitious matrix after it cracks, but tends to zero as the crack widens. For strain hardening materials, on the other hand, the residual stress increases after first cracking, and this may result in multiple cracking within the sample, which is then followed by failure at a localised crack.

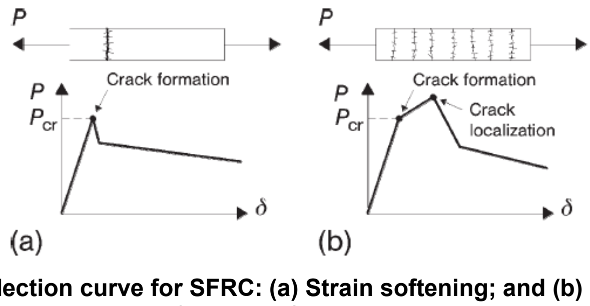



**Figure 2. Typical load-deflection curve for SFRC: (a) Strain softening; and (b) strain hardening behaviour (*fib* Model Code (2013)).**

The fib Model Code 2010 (2013) and DafStb Guidelines (2012) establish the residual flexural tensile properties of SFRC from three points notched prisms bending test, conducted in accordance with EN 14561 (2007). The determination of the residual tensile strength is done by multiplying consecutive factors with the residual flexural tensile strength obtained from the test. Since bending behaviour is markedly different from uniaxial tension behaviour, it may happen that softening materials in tension exhibit a hardening behaviour in bending (Figure 3).

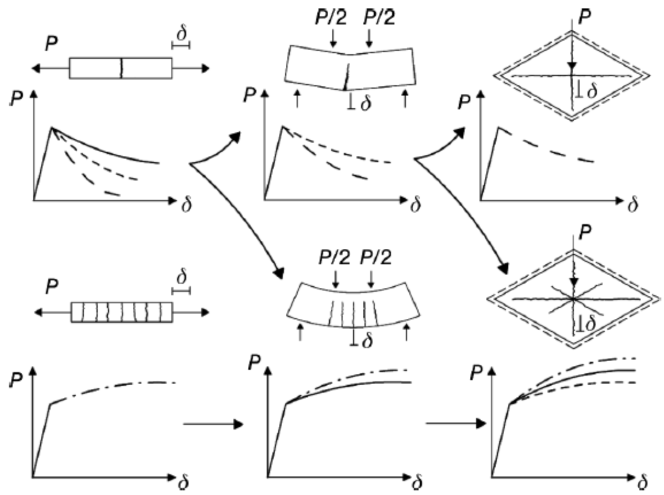



**Figure 3. Different response of structures made of SFRC having a softening or hardening behaviour under uniaxial tension or bending loads (fib Model Code 2010 (2013)).**

Traditionally, substantial amount of fibre dosage is required to produce a strain hardening or even a flexural hardening SFRC and it usually results in significant cost. However, in 2012, a global steel wire transformation company, NV Bekaert SA, has introduced a new series of steel fibres – tri-perfectly shaped end hooked fibres, commercially known as Dramix® 5D (Figure 4). The fibres are made of ultra-high tensile (2300 MPa) and ultra-high ductility (7% elongation capacity) cold drawn wire (BOSFA, 2017). Unlikely other conventional end hooked fibres where the fibres are expected to be deformed and pulled out of the cementitious matrix so as to provide the residual tensile strength and toughness, the tri-perfectly shaped end hooked fibres are engineered to provide perfect anchor; keeping the fibres firmly in place in the cementitious matrix and the pull-out mechanism is replaced by fibre elongation and providing the ductility on the same principle as conventional steel reinforcement. In the tests conducted at the University of New South Wales Australia, SFRC reinforced with a normal dosage of 25kg/m$^3$ of Dramix® 5D fibres demonstrated flexural hardening behaviour and, moreover, with 50kg/m$^3$ of Dramix® 5D fibres, the SFRC showed some strain hardening behaviour (Amin, 2015).

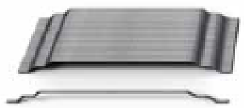


**Figure 4. New tri-perfectly shaped end hooked fibres (BOSFA, 2017).**

The fib Model Code 2010 (2013) and DafStb Guidelines (2012) suggest to characteristic the residual tensile strength of SFRC from the EN 14561 (2007) three point notched prisms bending test. While the NZS 3101 Part 2 (2006) also uses the three point notched prisms bending test, the conversion factors from residual flexural strengths to residual tensile strengths are different. The new AS 5100.5 Standard (2017) proposes a direct tension test to establish the residual properties of SFRC.

**SFRC FOR ULTIMATE LIMIT STATE DESIGN**

## Introduction

A major principle in construction is to create robust structures. Robustness is directly linked to the ductility of a structure. To prevent brittleness in structural elements, the fib Model Code 2010 (2013) suggests that steel fibres can be used to substitute conventional reinforcement at the ultimate limit state, if the following relationships:

$$f_{R1k} / f_{Lk} \geq 0.4 \quad (1)$$
$$f_{R3k} / f_{R1k} \geq 0.5 \quad (2)$$

where $f_{Lk}$ is the characteristic value of Limit of Proportionality determined using EN 14651 (2007) three point notched beam bending test and $f_{R1k}$ and $f_{R3k}$ is the characteristic residual flexural strength of SFRC corresponding to a crack mouth opening displacement (CMOD) of 0.5 mm and 2.5 mm, respectively.

### Determination of the Ultimate Bending Moment Capacity

The bending moment capacity of SFRC can be calculated using the fundamental SFRC constitutive laws. The ultimate limit state design of a cross-section for bending with or without axial force is based on following assumptions:

- Plain sections remain plain.
- The strain distribution is aligned with the strain distribution of reinforced concrete where:
  - The maximum strain the in extreme compression fibre, $\varepsilon_{cu}$, is taken as 0.003 (as per Australian Standards recommendation) to 0.0035 (as per NZS 3101 Part 2 (2006), Eurocode 2 (2004) and fib Model Code (2013))

In order to calculate the cross-section bending moment capacity, the static equilibrium needs to be determined. Figure 5 schematically represents the relation between stresses (and resultant forces) and strains, in line with the fib Model Code 2010 (2013) and DafStb Guidelines (2012). Instead of neglecting the resistance of concrete in the tension zone (i.e. concrete carries zero tension), the curvilinear post crack stress-strain relation of the SFRC is adopted so the steel fibres take a share in the tensile resistance. The bending failure stage is supposed to be reached when one of the following conditions applies:

- attainment of the ultimate compressive strain in the SFRC, $\varepsilon_{cu}$
- attainment of the ultimate tensile strain in the steel (if present), $\varepsilon_{su}$
- attainment of the ultimate tensile strain in the SFRC, $\varepsilon_{Fu}$, which is taken as a function of ultimate crack width.

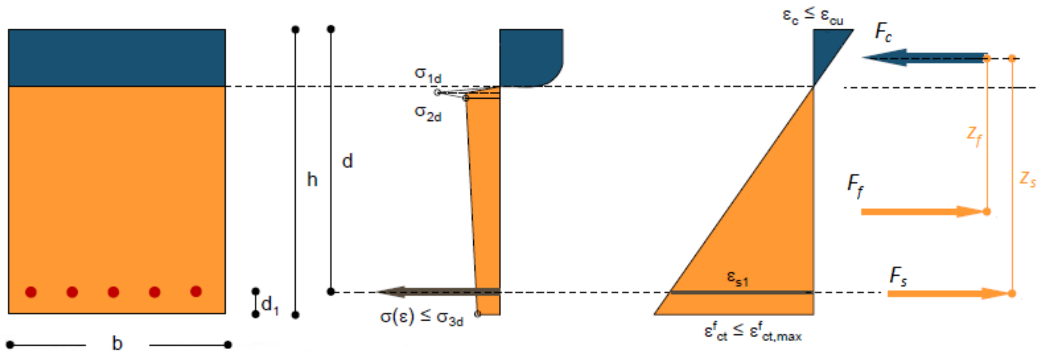



**Figure 5. Static equilibrium of the cross section under bending.**

From Figure 5, the ultimate bending moment capacity, $M_u$, of the cross section can be written as follows:

$$M_u = F_f \times z_f + F_s \times z_s \quad (3)$$

AS 5100.5 Standard (2017) has proposed a simplified stress blocks shown in Figure 6. In this case the contribution of the fibres is taken to be plastic with a constant stress at 1.5 mm crack opening distance of $f'_{1.5}$ applied to the section on the tensile side of the neutral axis. Forces and moments are resolved using equilibrium and compatibility in the usual way as per Equation 3.

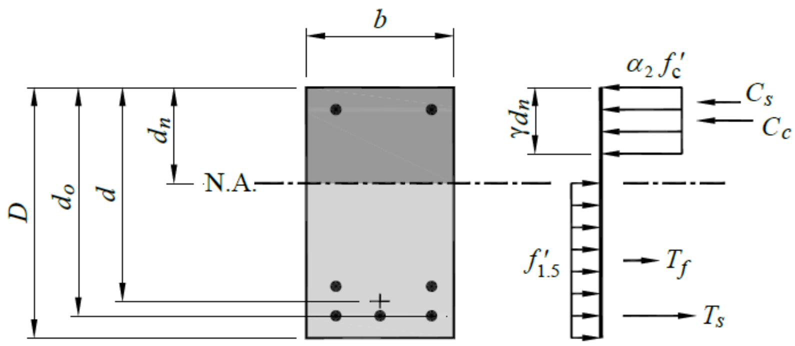



**Figure 6. Design for bending in accordance with AS5100.5 (2017).**

NZS 3101 Part 2 Standard (2006) uses similar approach with the fib Model Code 2010 (2013) and DafStb Guidelines (2012); however, the standard has also included a size-dependent safety factor, $\kappa_h$, in line with the RILEM Technical Committee 162 recommendation. At the time of the RILEM recommendation being published, Schnütgen and Vandewalle (2003) reported that that the origin of $\kappa_h$ is not yet fully understood and proposed to undertake further investigation. The RILEM recommendation and NZS 3101 Part 2 relate the crack width to the member section size and the tensile strain in SFRC. While the $\kappa_h$ scaling factor is necessary, the proposed values are not correctly derived and resulted in overly conservative bending moment capacity.

To illustrate this, considers a singly reinforced concrete slab. The slab is made of 35 MPa grade concrete, reinforced with 12 mm diameter grade 500E bars spaced at 200 mm centres, 30 kg/m$^3$ of the tri-end hooked fibres and the cover to reinforcement is 40 mm. Figure 7 compares the ultimate bending moment capacities of the slab, for various thicknesses, predicted using the NZS 3101 Part 2 (2006), *fib* Model Code 2010 (2013), and AS 5100.5 (2017). As can be seen, the NZS 3101 Part 2 (2006) is significantly conservative when the member thickness exceeds 200 mm.

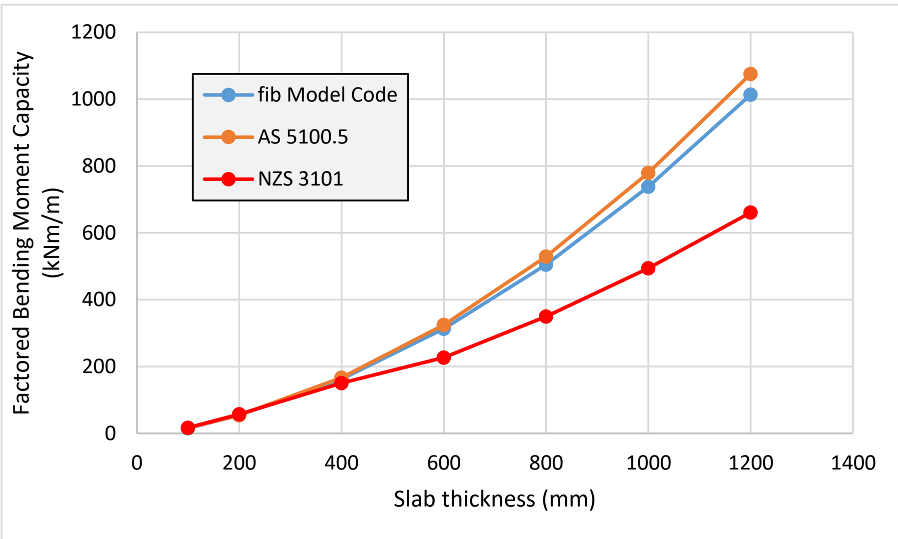



**Figure 7. Ultimate bending moment capacities predicted using**
**NZS 3101 Part 2 (2006), fib Model Code 2010 (2013) and AS 5100.5 (2017).**

In the case of structural elements, subjected to bending, where steel fibres completely replace the conventional reinforcement, a minimum redundancy level is required. This residual post crack strength of the SFRC becomes significant as a remarkable stress redistribution must occur in order to achieve the required ductility. It is obvious that a flexural strain softening SFRC cannot be used in this case; the fact is that the flexural tensile strength of the uncracked SFRC is higher than the flexural tensile strength of the cracked SFRC. This mean that 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 structure will collapse, i.e. the first crack is the last crack. Of course, this applies to plain concrete as well.

For this reason, in addition to the limitations provided by Equations (1) and (2), SFRC used in structural elements without conventional reinforcement, subjected to bending, must at least have a flexural hardening behaviour, if not strain hardening behaviour. Once the SFRC is cracked, the flexural hardening SFRC can take a higher flexural strength.

**Determination of the Ultimate Shear Capacity**

The effect of steel fibres onto the shear and punching shear resistance can be taken into account by an additional component in the respective equations. Steel fibres act like a shear reinforcement over the entire cross section of the structural element. The shear capacity of the element is increased as a function of the performance of the SFRC used. According to *fib* Model Code 2010 (2013) and the DafStb Guidelines (2012), this can lead to a significant reduction (or even a complete elimination) of conventional shear reinforcement. Whilst the shear design models in the NZS 3101 Part 2 (2006), *fib* Model Code 2010 (2013) and the DafStb Guidelines (2012) are different, the codes take the increased shear resistance due to SFRC into account by introducing an additional element, $V_{fd}$, into the equations for conventional shear design, which can be written as:

$$V_{rd,3} = V_b + V_{wd} + V_{fd} \tag{4}$$

where $V_{rd,3}$ is the ultimate shear capacity, $V_b$, $V_{wd}$ and $V_{fd}$ is the design shear strength contributed by the concrete matrix, stirrup and the fibres, respectively. Equation 4 holds for structural elements with and without conventional shear reinforcement.

AS 5100.5 Standard (2017) adopts the shear model based on the simplified modified compression field approach and the total ultimate shear capacity can also be estimated using Equation (4). For the initial implementation of the standard, some additional rules are adopted. Recently, Foster et al. (2017) have proposed an updated version of the shear model based on the simplified modified compression field approach. Space in this paper prohibits an extensive review on the shear design methodology. Readers are referred to appropriate references when more detailed information is sought.

## SFRC FOR SERVICEABILITY LIMIT STATE DESIGN

The strain softening behaviour of SFRC is problematic in terms of calculating crack widths. Although it is theoretically possible to calculate a crack width in a section that has a permanent compression zone, the fact is that the tensile strength of the uncracked fibre reinforced concrete is higher than the tensile strength of the cracked fibre reinforced concrete. This means that for a concrete element where the full section is in tension, for example due to restraint of shrinkage and temperature stresses in a ground slab, the cracked section is the weakest section and it is not possible to determine accurately if and where the concrete section will crack again i.e. it is impossible to determine a theoretical spacing between cracks and without a crack spacing it is also impossible to determine a crack width using current crack width calculation theory.

When conventional and steel fibre reinforcement are combined the strain softening behaviour of SFRC does not change. However, the post cracking tensile capacity of the SFRC can be taken into account when calculating crack widths for the conventional reinforcement. The basic principle is that due to increasing post crack strength the released force at crack formation decreases: The fibres carry a part of the released force. As a consequence, the reinforcing steel needs to transfer only a reduced force back into the concrete. Therefore, the strain in the reinforcing steel, as well as the required transfer length, is directly reduced. For a given crack width, the use of steel fibres can thus significantly decrease the required amount of conventional reinforcement. Additional effects from enabling the use of smaller diameters can be utilised.

NZS 3101 Part 2 (2006) and *fib* Model Code 2010 (2013) provide a "deemed-to-comply" formulation in order to obtain controlled crack formation in a SFRC element. The AS 5100.5 (2017) has adopted a modified version of the equation in NZS 3101 Part 2 (2006) and *fib* Model Code 2010 (2013). Schnütgen and Vandewalle (2003) suggested that the resulting crack width is approximately 0.25 mm if a 1.4 reduction factor is applied to both the residual tensile stress of the SFRC and the maximum stress permitted in the steel reinforcement.

The crack width design approach corresponds to the method for reinforced concrete introduced in Eurocode 2 (2004). The fundamental reinforced concrete crack width design equation are shown below.

$$w_k = s_{r,max}\,(\varepsilon^f_{sm} - \varepsilon_{cm}) \qquad (5)$$

where: $s_{r,max}$ = maximum crack spacing in a combined steel fibres and conventional reinforced concrete

$(\varepsilon^f_{sm} - \varepsilon_{cm})$ = the difference between the mean strain in the reinforcement and the mean strain in the concrete

Following the DAfStb Guidelines (2012), the rules of Eurocode 2 (2004) are amended by the post crack tensile strength provided by the SFRC. This is done by introducing $\alpha_f$ as the ratio of the post crack tensile strength over the first crack tensile strength. As a simplification of this concept the crack width $s_{r,max(SFRC)}$ and $(\varepsilon^f_{sm} - \varepsilon_{cm})_{(SFRC)}$ of SFRC may be calculated as:

$$s_{r,max\,(SFRC)} = (1 - \alpha_f)\,s_{r,max} \qquad (6)$$

$$(\varepsilon^f_{sm} - \varepsilon_{cm})_{(SFRC)} = (1 - \alpha_f)\,(\varepsilon^f_{sm} - \varepsilon_{cm}) \qquad (7)$$

As can be seen in Equations (5) to (7), the $(1 - \alpha_f)$ factor can also be applied to other crack width prediction models. It is critical to note that the $(1 - \alpha_f)$ factor shall be applied to both crack spacing model and the strain prediction model between the steel reinforcement and concrete.

## MINIMUM FIBRE DOSAGE RECOMMENDATIONS

The performance of SFRC generally increases with increasing fibre dosage. However, it is not practical, not economical and not sustainable to specify an excessive high dosage of fibre in a concrete for which the extra dosage is not structurally required.

An over-dosage of steel fibres also results in decrease in SFRC workability, increase the risk of fibre balling and, more importantly, up to a certain fibre dosage, the addition of fibre dosage does not further improve the SFRC performance due to the weaker cementitious matrix and crack paths find ways of minimum resistance and are likely to divert around fibre ends (Foster et al., 2013).

On the other hand, an absolute minimum fibre dosage shall be specified to ensure minimum overlap between fibres and provide consistency network of fibres in the concrete.

Therefore, the fibre dosage of SFRC are governed by the maximum of:

(i) Minimum fibre dosage for ensuring the required SFRC performance; and
(ii) Minimum fibre dosage based on minimum overlap.

### Minimum Fibre Dosage for Ensuring the Required SFRC Performance

For structurally designed application, the minimum fibre dosage is to satisfy the limit states design requirements as discussed in Sections above. Design engineers should be aware that not all fibres perform equally. The required fibre dosage varies depending on the steel fibre types and products. Hence, the design engineer must have a sufficient level of confidence that the specified fibre product and fibre dosage can satisfy the design properties; this can be achieved by either undertaking the EN 14651 (2007) three point notched beam bending test and/or, if available, using the steel fibre manufacturers and suppliers' data sheet.

**Minimum fibre dosage based on minimum overlap**

Based on fibre spacing theory (Figure 8), McKee (1969) suggested that the average distance between fibres, *s*, can be estimated as:

$$s = \sqrt[3]{\frac{\pi \times d_f^2 \times l_f}{4\,\rho_f}} \qquad (8)$$

where $l_f$ is the length of the fibre, $d_f$ is the diameter of the fibre and $\rho_f$ is the percentage of fibre by volume.

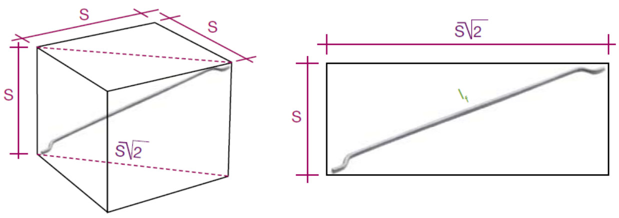



**Figure 8. Minimum dosage based on minimum overlap concept.**

The European Standard for Sprayed Concrete EN 14487-1 (2006) suggested that the average distance between steel fibres, *s*, should be lower than 0.45 of the fibre length, $l_f$, in order to ensure a minimum overlap between fibres. While sprayed concrete is generally used for passive tunnel lining protection and light structural and bearing applications, for structurally designed applications, it is recommended that *s* should be lower than 0.4 $l_f$, as adopted by AS 5100.5 (2017).

## QUALITY CONTROL OF SFRC FOR STRUCTURAL APPLICATIONS

Quality assurance is fundamental in SFRC construction so as to provide safe and durable structures. The quality control process shall involve all parties working in the project; noting that it may be too risky to rely on the promises of a manufacturer alone. Table 1 lists the responsibility of each party.

**Table 1. Quality control responsibility of each party in manufacturing SFRC**

| Party Involved | Responsibility |
|---|---|
| Fibre manufacturer or supplier | Ensure the fibres are complied with EN 14889-1 (2006) CE Marking Class 1 and have a minimum level of quality and performance |
| Ready mix concrete company | Ensure the correct type of fibre with the correct fibre dosage is batched. |
| Engineer and/or contractor on site | Check the correct type of fibre with the correct fibre dosage is batched and ensure the fibres are uniformly distributed in the concrete. |

**Steel fibre material quality – EN 14889-1: Fibres for concrete – Part 1: Steel fibres**

The Australian Standard AS 5100.5 (2017) requires all steel fibres to be complied with EN 14889-1 (2006) CE Marking Class 1. Likewise, the UK Concrete Society Technical Report No. 34 (2014) also requires the fibres used in slab and pavement construction to be manufactured in accordance with EN14889-1 (2006).

EN 14889-1 (2006) is the European quality control performance based manufacturing standard for steel fibres. It is mandatory in European Union member states for steel fibres used in construction to be manufactured in accordance with this standard. There are two types of classification, Class 1 for structural use and Class 3 for non-structural use. The term "structural use" is where the addition of fibres is designed to contribute to the load bearing and carrying capacity of the concrete element including pavements and slabs on grade. For this reason, Class 1 steel fibres are submitted to more scrutiny during manufacture (more intensive sampling and testing) and production is monitored by an external third party. Other standards, such as ASTM A 820 (2016), do not necessarily require third party verification and are not performance based. Hence, EN 14889-1 (2006) standard can provide engineers a higher level of confidence that the fibres have a minimum level of quality and performance.

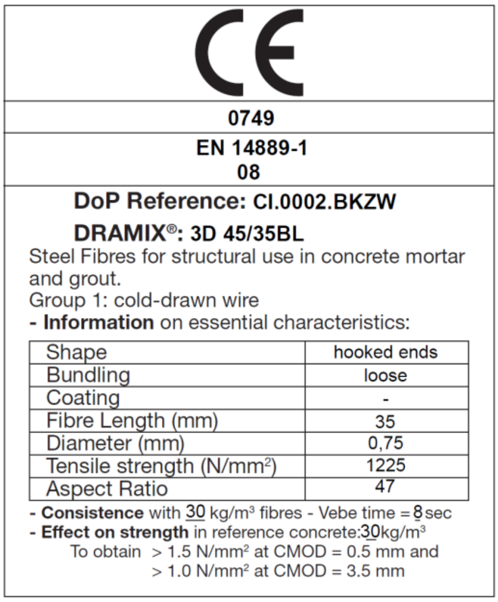
By adhering to the standard, manufacturers are required to:

- Class their fibre in accordance with the base material; cold drawn wire, cut sheet, melt extract, shaved cold drawn wire or milled from blocks and then declare the shape; straight or deformed. This allows any steel fibre to be manufactured in accordance with this standard, provided it can be produced within the control and tolerances set to guarantee quality and consistency;
- Declare values for each individual fibre characteristics that influences performance; such as length, diameter, aspect ratio, fibre tensile strength, etc and these values must not deviate by more than the tolerances specified in the standard;
- Declare a minimum fibre dosage to meet
  (i) A level of consistency or workability; and
  (ii) A prescribed residual flexural strength values in a reference concrete.

  This enables complete transparency allowing the engineer, concrete company, and contractor to legitimately compare the expected performance of different fibre types on offer.

  *(It is important to note that nowhere in the standard a minimum fibre dosage has been defined. The minimum fibre dosage shall be determined using Equation (8) in Section above.)*

Every packaging of steel fibres that complied with EN 14889-1 (2006) has a CE label attached. A typical example is shown in Figure 9. The concrete company can use this information to record that the correct fibre type has been used in the supply of their SFRC.



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EN 14889-1
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**DoP Reference: CI.0002.BKZW**

**DRAMIX®: 3D 45/35BL**

Steel Fibres for structural use in concrete mortar and grout.
Group 1: cold-drawn wire
**- Information** on essential characteristics:

| | |
|---|---|
| Shape | hooked ends |
| Bundling | loose |
| Coating | - |
| Fibre Length (mm) | 35 |
| Diameter (mm) | 0,75 |
| Tensile strength (N/mm$^2$) | 1225 |
| Aspect Ratio | 47 |

**- Consistence** with 30 kg/m$^3$ fibres - Vebe time = 8 sec
**- Effect on strength** in reference concrete: 30kg/m$^3$
To obtain > 1.5 N/mm$^2$ at CMOD = 0.5 mm and
> 1.0 N/mm$^2$ at CMOD = 3.5 mm

**Figure 9. Example of CE Label.**

## Quality Control in SFRC Production

Steel fibres can be introduced into the concrete manually or through an automatic dosing equipment. The automatic dosing equipment can be linked to the central batching system, which allows accurate dosing and provides a record for quality control documentation.

In Australia and New Zealand, it is still a common practice that steel fibres are batched manually at the concrete batching plants. For quality control in the SFRC production, the following steps shall be followed by the fibre batcher:

- Before batching and loading the fibres into the concrete truck, the fibre batcher shall check the CE label on the fibre package against the delivery docket and circles the fibre type on the CE label if they match.

- After batching and loading the fibres into each concrete truck, the fibre batcher manually counts the number of the empty bags and writes the quantity of bags and the bag size on the delivery docket.

**In-situ Fresh SFRC Quality Inspection**

A visual inspection is the common practice to determine whether random distribution and the separation of collated fibres has been achieved. Balling of steel fibres shall be avoided. At the same time, the concrete shall also be inspected to check the correct type of fibre is being used.

To quantify the fibre dosage and homogeneity in the fresh concrete, test shall be carried in accordance with EN 14721 (2007) Method B. Each test is made up of three samples of at least 10 litres in volume, one in each third of the same load as follows:

(i) at the beginning of the load (from the first third of the mix), after 0.5 $m^3$ is unloaded;
(ii) in the middle of the load (from the second third of the mix); and
(iii) at the end of the load (from the last third of the mix), with minimum 0.5 $m^3$ left in truck.

All samples shall be taken directly out of the “concrete stream” at the end of the chute and not out of a wheelbarrow as it may give segregation.

This method has been adopted by the AS 5100.5 (2017) and South Australian Water Corporation (SA Water) Technical Standard TS 710 – Concrete (2016).

**Confirmation Testing of SFRC Performance**

Confirmation testing of the post crack flexural or tensile strength using the EN 14651 (2007) three point notched bending test is to obtain the first-hand information of the material properties of the concrete actually used. However, testing cannot be carried out on-site and results can only be expected after 28 days, as the concrete has to be cured and gained strength. A 28 days waiting period represents a significant cost and delay to the construction program. Further, in Australia and New Zealand, only limited accredited laboratories that can perform the EN 14651 (2007) test and it is sometimes not feasible to carry out the test. For this reason, project quality control plan can be developed as follows:

(i) An initial or pre-construction post crack flexural strength test using EN 14651 (2007) is first be carried out; and
(ii) Throughout the project, the confirmation testing of post crack strength may be replaced by testing of both first crack flexural strength and the fibre dosage. If first crack flexural strength is within 10 % of the initial type testing, and provided that the concrete mix design has not changed and fibre content is sufficient, the post crack strength can be assumed to be satisfactory as according to OVBB (2008) and DafStb Guidelines (2012), the first crack strength and post crack strength are closely correlated if the same type of fibre is used in the same type of concrete.

## SFRC FOR STRUCTURAL APPLICATIONS

A number of projects have been constructed in countries all over the world using SFRC for structural applications, some where flexural hardening SFRC’s were used without conventional reinforcement while others used combined SFRC and conventional reinforcement.

**Komatsu Heavy Machinery Warehouse, Port Hedland, Western Australia**

The Komatsu heavy machinery warehouse in Port Hedland is located within one of the most severe cyclonic wind regions in Australia, being Region D Terrain Categeory 2. Due to the high wind load pressure, the footing needs to resist a design bending moment of 1250 kNm/m. The original design was 1.5 m thick with N28 conventional reinforcement at 200 mm centres. Being located at 1500 km north of Perth city, transporting conventional reinforcing cage to Port Hedland is very costly and has a long lead time. Consequently, the contractor has selected to use 30 kg/$m^3$ of the tri-end hooked flexural hardening SFRC to totally replace all the reinforcement as the SFRC has a bending moment capacity in excess of the design moment requirement. As a result, construction was simplified, sped up and the contractor was able to reduce construction time by 4 weeks. Significant amount of savings were achieved; not only due to the shortening of the construction time, but there are also no transportation, fabrication and on-site placement cost of the conventional reinforcement. A concrete pump was also not needed as the truck mixers could discharge from the chute directly at the pouring point.

**Major Waste Transfer Station, Sydney, Australia**

A combined duo- end hooked SFRC and light conventional reinforcement was used for the construction of Veolia waste transfer station 12000 $m^2$ external apron slabs and hardstands and 2000 $m^2$ internal waste transfer processing slab. The entire external and internal slabs are joint free. The external hardstand is designed as 260 mm thick and is capable for withstanding loads from the 100t axle load Container fork lift and containers. The internal waste transfer processing slab is designed to be watertight (crack width of 0.15 mm) and is constructed with high strength concrete, up to 90 MPa, in order to handle very aggressive domestic waste with low PH level. Conventional design would require control joints at a regular intervals with acid resistance liquid-stops. The combined duo- end hooked SFRC and light reinforcement totally eliminate the needs for control joints and hence the expensive liquid-stop.

**Figure 10. Komatsu Heavy Machinery Warehouse Foundation reinforced with steel fibre only.**

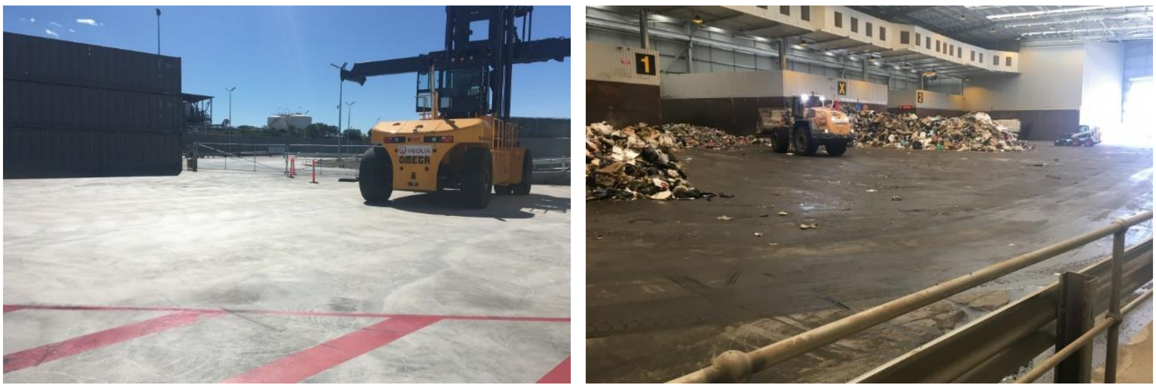

**Figure 11. SFRC and conventional reinforcement for Veolia Major Waste Transfer Station, Sydney, Australia.**

**Pile caps and capping beams for new manufacturing and warehousing facility in New Zealand**

SFRC was used as a fast and cost effective option for the foundations of a new manufacturing and warehousing facility in New Zealand. Using the flexural hardening SFRC, it totally replaces all the 12 mm diameters at 100 mm centres conventional reinforcement in the pile caps and ground capping beams.

**Kiwi Rail Locomotive Wash Raft Slab in New Zealand**

The new Kiwi Rail Locomotive wash slab in Christchurch, New Zealand, was designed to be watertight without any joints. The raft slab is generally 200 mm thick and needs to resist a design moment 45 kNm/m. Using 35kg/m$^3$ of the tri-end hooked SFRC together with light conventional reinforcement, i.e. 16 mm diameter bars at 250 mm centres spacing, it provides an ultimate bending moment capacity of more than 65 kNm/m and the combined fibres and mesh were also used in the slab for achieving a nominal crack width of 0.2 mm. The solution results in lesser demand in conventional reinforcement, which made it easier and quicker to construct, and, therefore, more cost effective.

**Figure 12. SFRC Pile caps and capping beams without conventional reinforcement (right photo shows the original designed but since abandoned reinforcing cage).**


**Figure 13. SFRC and conventional reinforcement for Kiwi Rail Locomotive Wash Raft Slab**

**Clad rack foundation: IKEA High Bay Warehouse, Sydney, Australia**

A clad rack building is a building consisting of a structural racking system. The slab acts as the foundation for the walls, roof and the racks. Doing so, the slab foundation structure is not only carrying the stored goods but must also withstand external loads like wind load and earthquake load. The clad rack foundation for the IKEA High Bay Warehouse building in Sydney, has a height of over 30 m. Three important load cases are mainly determining the slab design:

(i) the situation where heavy wind is present on the moment that the racks are empty;
(ii) the situation during a seismic event on the moment that the racks are empty; and
(iii) the situation where the racks are fully filled with goods.

The slab is 300 mm thick in general and the edges were thickened to 450 mm thick in order to resist the forces and moments transferred from the structural racking bracing towers at the edge. The entire slab area is in excess of 10000 $m^2$ and was designed as a monolithic slab, i.e. without any movement and control joints. To meet both serviceability crack control and ultimate limit state requirements the slab is reinforced with 35 kg/m³ of the duo-end hooked fibres, a top mesh consisting of 9 mm diameter bars at 125mm centres, both ways. At the edge thickening section, a bottom mesh consisting of 9 mm diameter bars at 125mm centres, both ways were also placed.

## CONCLUSIONS

Structural steel fibre reinforced concrete applications differ mainly from well-known fibre applications like floors and pavements. In structural applications, steel fibres are the main or the secondary reinforcement to take up bending moments and the shear stresses. This paper presents some basic principles governing the structural design of SFRC based on the available design codes. Starting from classification of various types of steel fibres, which have profound effect on the performance of concrete structures, the simple constitutive models are presented, allowing engineer and designer to design with SFRC. Numerous projects have been carried out. A few of them are presented, giving insight information on where steel fibre only reinforcement and combined steel fibre and conventional reinforcement were used.

**Figure 14. Clad rack building for IKEA, Sydney Australia.**

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