OPTIMIZATION OF FLOWABLE CONCRETE FOR STRUCTURAL DESIGN: PROGRESS REPORT OF fib TASK GROUP 8.8

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ABSTRACT

With the tendency to apply concrete with a higher workability and the use of new concrete components more options are available to design concrete. New concrete types like self-compacting concrete (SCC), ultra-high performance fibre reinforced concrete (UHPFRC) and high performance fibre reinforced cementitious composite (HPFRCC) also require adopted mix designs and in case, a new engineering approach. Flowable concrete not only is an innovative material, but its application can lead to innovative structures, combinations of building materials and alternative and efficient production methods. fib Task Group 8.8 aims at facilitating the use of innovative flowable materials for the design of concrete structures and considers three aspects of flowable concrete for structural design: material properties, production effects and structural boundary conditions. The main objectives of this group of experts are to combine research findings with practical experience and to write a state-of-the-art report and a recommendation on the structural design with flowable concrete. Areas of structural design where flowable concrete differs from traditional vibrated concrete have to be identified. This paper reports about the progress of fib TG 8.8 and discusses important aspects related to flowable concrete.

Key-words: Flowable concrete, material behaviour, recommendation, test method.

RECOMMENDATIONS FOR FLOWABLE CONCRETE

In order to obtain a highly flowable concrete (self-compacting or compacted with some vibration), the mixture composition has to be adjusted, which can have consequences related to properties in the hardening and hardened states. Due to the flowable nature, reinforcing bars can become an obstacle, mixture components may float or segregate and the casting technique determines the orientation of fibres, if applied. Research is ongoing on many topics addressed by commission fib TG 8.8.

Recent standards cover different concrete types only for a part. With the publication of the Model Code 2010 (MC2010) [1,2] many improvements have been achieved and a wider area of flowable concrete is now covered. fib Bulletin 70 'Code-type models for concrete behaviour' [3], which is currently under preparation, describes the background of MC2010 without considering fibre reinforced concrete; fib TG 8.3 (Fibre reinforced concrete) works on a document to fill this gap. MC2010 extends the range of compressive strength classes up to C120/140 (characteristic cylinder compressive strength of 120 MPa) and offers provisions for the design of structural elements with fibres in bending and shear, which allows gaining experience in the following years. Furthermore, the K-concept has been implemented, which connects experimental testing on a small scale (test specimens) with the structural behaviour. In the future, it is expected that flow simulations and further research will allow a better prediction of the structural behaviour. The extension of MC2010 related to strength classes does not mean that already sufficient experience is available in the complete area covered by MC2010. Furthermore, MC2010 limits the concrete strength to C120/140; whereas the French UHPFRC-recommendation [4] requires a minimum characteristic compressive strength of 150 MPa. A gap is left for strength classes in the range of 120 to 150 MPa. Figure 1 maps main areas of experience with fibre reinforced concrete. It is important to gain experience with mixtures not yet covered by standards, to compare their performance with more common concrete types and to develop reliable design standards.

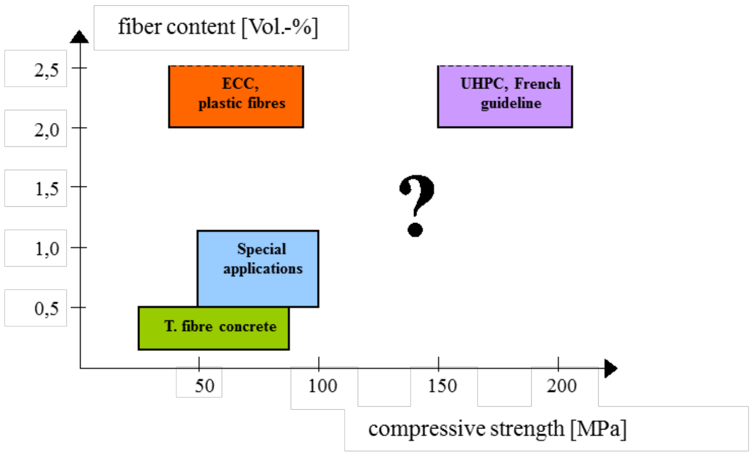


Figure 1 - Range of different concrete types concerning compressive strength and fibre content.

Two important published documents for flowable concrete are the French UHPFRC-recommendation [4] and the Japanese HPFRCC-guideline [5] covering ultra-high strength concrete and tensile-strain hardening composites, respectively. A significant amount of experience with UHPFRC is documented in the recently published French UHPFRC-recommendation. UHPFRC according to this recommendation has to have a minimum characteristic cylinder compression strength of 150 MPa and has to fulfil a criterion on minimum ductility. The document is adjusted to the lay-out of Eurocode 2 and the provision for shear capacity has been further validated with experimental data. The fatigue behaviour of UHPFRC was identified as an important research field.

TEST METHODS FOR FLOWABLE CONCRETE

The need for practicable and reliable test methods for fibre reinforced concrete is obvious. Especially, for quality control during production such test methods are lacking or have important drawbacks. Recently, tests on cubes were proposed (destructive and non-destructive) that allow for the qualification in three different directions (i.e. Multidirectional Double-Punch Test [6] and the Double Edge Wedge-Spitting Test [7]) taking into account both the distribution and the orientation of fibres. Herein, mainly steel fibres are considered and tested. The translation of experimental results of small specimens to larger structures (Figure 2) has to be executed with care. The focus should be on the prescription concerning manufacturing and representativeness of the obtained orientation with respect to the intended application. A homogenous fibre distribution (within reasonable boundaries) is a necessity for adequate design assumptions.

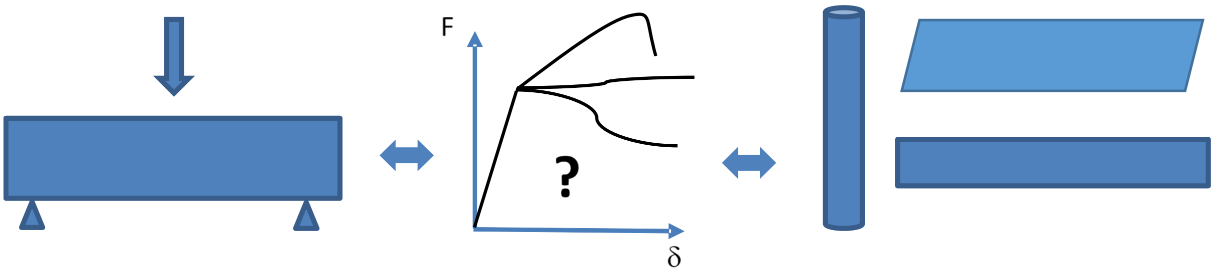


Figure 2 – Translation of results of test specimens to the performance of full-scale structures.

Standardized specimens

As the discussion indicates, for the translation of results obtained with small scale specimens (i.e. prisms) to a concrete structure several parameters have to be considered. A more favourable orientation of fibres can result in an overestimation of the structural performance, whereas a large scatter and an orientation less favourable compared to a structure can result in an underestimation. Ferrara and Cresmonesi [8] studied the use of a funnel as a tool to standardize the flow through prisms. The concrete had a slump flow of 680 mm after mixing (the fibre length of the hooked-end steel fibres was 35 mm (fibre dosage: 50 kg/m^3; maximum aggregate size: 8 mm). Thirteen beams (150·150·600 mm^3) were cast (9 from the centre and 4 from an end of the mould). Three-point bending tests were executed with notched specimens according to EN 14651 [9]. Higher bending stresses were observed for specimens filled from the centre of the mould (Fig. 3a). The number of fibres in the centre (1.45 fibres/cm^2 cast in the centre compared to 1.12 fibres/cm^2 cast at the end of the mould) indicates that some separation occurred at the point of casting where higher shear rates and stresses are present. Figure 3b compares the fibre orientation factor α with the maximum stress σ_N of the flexural tests.

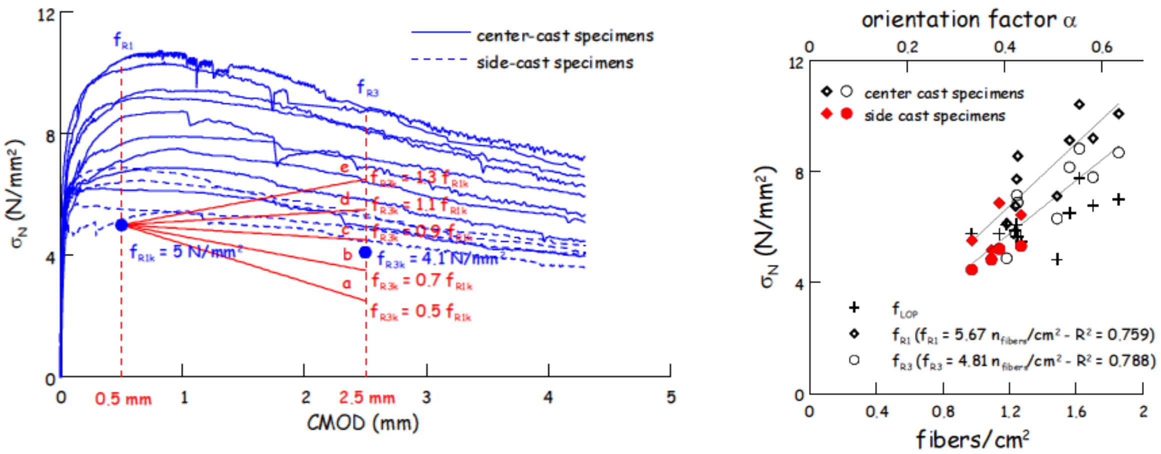


Figure 3 – Three-point bending tests a) nominal stress – CMOD curves and material classification and b) bending stresses versus specific number of fibres and fibre orientation factor.

MATERIAL CHARACTERISTICS

Hereafter, a selection of material characteristics will be discussed. The selection reflects the variety of concrete types considered as 'flowable concrete'. Because of the wide range of mixture compositions and characteristics, the modulus of elasticity is expected to be in a wide range at a given compressive strength. Figure 4 compares provisions given in MC2010 [1], the German DAfStb-document on UHPFRC [10] and the Japanese HPFRCC-guideline [5]; Table 1 shows the equations.

Table 1 – Equations to calculate the modulus of elasticity with reference

Equation	To be used for	Reference
$E_{ci} = E_{c0} \cdot \alpha_E \cdot \left(\frac{f_{ck} + \Delta f}{10} \right)^{1/3}$	Basalt, dense limestone aggregates, αE=1.2 Quartzite aggregates, αE=1.0 Limestone aggregates, αE=0.9 Sandstone aggregates, αE=0.7	[1]
$E_{HPFRCC} = 1.77 \cdot 10^4 \cdot \sqrt{\frac{\gamma}{18.5}} \cdot \left(\frac{f'_{ck}}{60} \right)^{1/3}$		[5]
$E_C = 9500 \cdot fc^{1/3}$	Coarse grain UHPFRC	[10]
$E_C = 8800 \cdot fc^{1/3}$	Fine grain UHPFRC	[10]

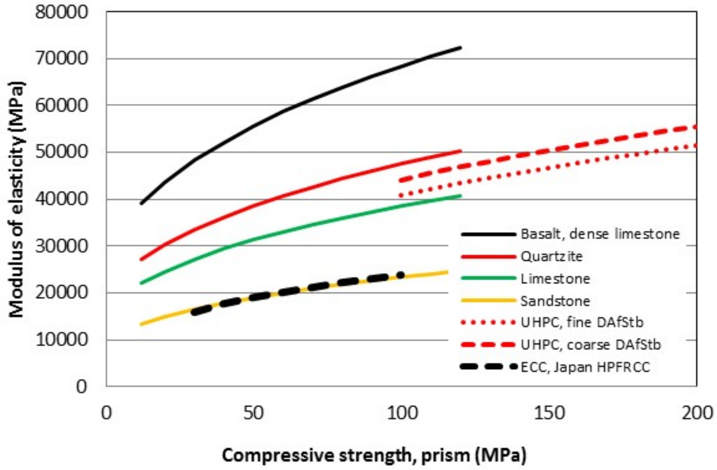


Figure 4 – Comparison of the theoretical modulus of elasticity according to three recommendations (Model Code 2010 [1], DAfStb UHFB [10] and Japanese HPFRCC–guideline [5]).

The applicable range of compressive strength is defined only by MC2010 (≤ C120/140). Considering Quartzite as a common aggregate type in concrete, the modulus of elasticity of UHPFRC (coarse grain [10]) is following this trend quite well. Concrete with basalt aggregates, especially for higher strength classes significantly overestimates the related modulus of elasticity. On the other hand, HPFRCC can have a modulus of elasticity comparable with lightweight concrete or normal weight concrete with sandstone aggregates (Fig. 4). The paste content of HPFRCC is much higher and the maximum aggregates size is much smaller (typically not more than 0.5 mm) compared to traditional vibrated concrete. Therefore, a lower modulus of elasticity is expected. Flowable concrete can contain a high amount of fine grains. In addition, in order to obtain a high degree of packing and/or a high strength often very fine powders like microsilica are added, which results in a fast (thixotropic) structural build-up at rest. According to Roussel [11], three categories of thixotropy can be distinguished for SCC (Table 2).

Table 2 – Classification of SCC according to the flocculation rate [11]

Flocculation rate Athix (Pa·s)	SCC type
Less than 0.1	Non–thixotropic
Between 0.1 and 0.5	Thixotropic
Higher than 0.5	Highly–thixotropic

Thixotropy can cause problems related to the formation of casting layers and concrete aesthetics. However, the structural build-up also can be exploited to improve the production efficiency like in the case of slip-cast paving [12] or for the production of panels with a flexible mould [13], see also Figure 5. When casting self–levelling concrete that rapidly builds up strength, the mould can be removed (or deformed) quickly, which can save considerable production time.

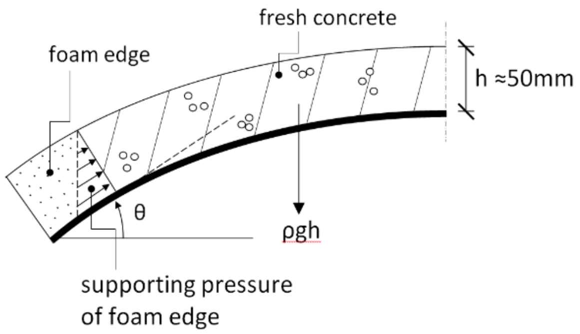


Figure 5 – Self-supporting concrete in a flexible mould utilising the thixotropic increase of the yield value as a production tool [13].

Concrete flow in areas of high shear stress

Shear- and gravity-induced migration of particles can lead to local differences in concentrations. Spangenberg et al. [14] studied the distribution of coarse aggregates (depending on the distance from the casting point and the vertical position) in beams cast with SCC (dimensions of beam: L=4.0/H=0.3/W=0.2 m). Different coarse aggregate concentrations were observed over the height of the beam. The value at the highest point did not indicate segregation (Fig. 6a). However, at a height of about 120 mm, the concentration was lower compared to other areas of the beam. According to the authors, differences in shear rate and stress cause rheological differences of the concrete, which promotes the migration of particles (Fig. 6b). The migration of particles increases the yield stress and viscosity in the lower layer until a state of balance is obtained. Simulations indicated that the highest shear rate was obtained at a height of 50 to 100 mm from the bottom.

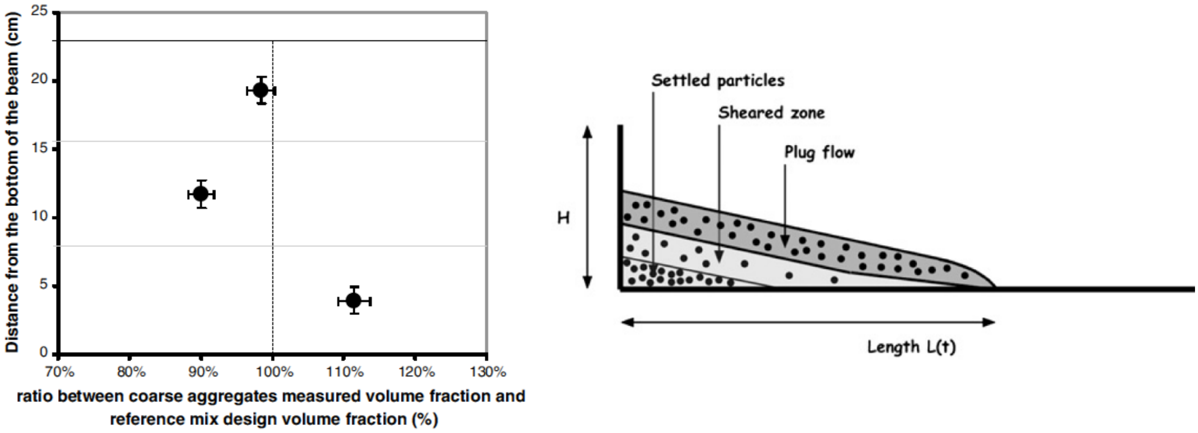


Figure 6 - a) Distribution of coarse aggregates across the height of a beam cast with SCC and b) particle migration in areas of high shear rates (close to the bottom of the mould) [14].

Fibre efficiency

The post-cracking behaviour of fibre reinforced concrete is correlated with the number of fibres crossing a crack, but also with the contribution of the fibres, which depends on their orientation and efficiency. Neighbouring fibres can decrease the efficiency of a fibre compared to the single fibre pullout test. Sato et al. [15] carried out deformation-controlled tests in compression and uni-axial tension. Based on a reference concrete, the effect of the type (straight and hooked-end), the dosage and combinations of steel fibres was studied. Uni-axial tensile strengths up to 30 MPa were obtained. Figure 7a shows the maximum strength in compression. In uni-axial tension (Fig. 7b), the results of combinations of fibres (white dots) followed a linear relationship with the fibre factor.

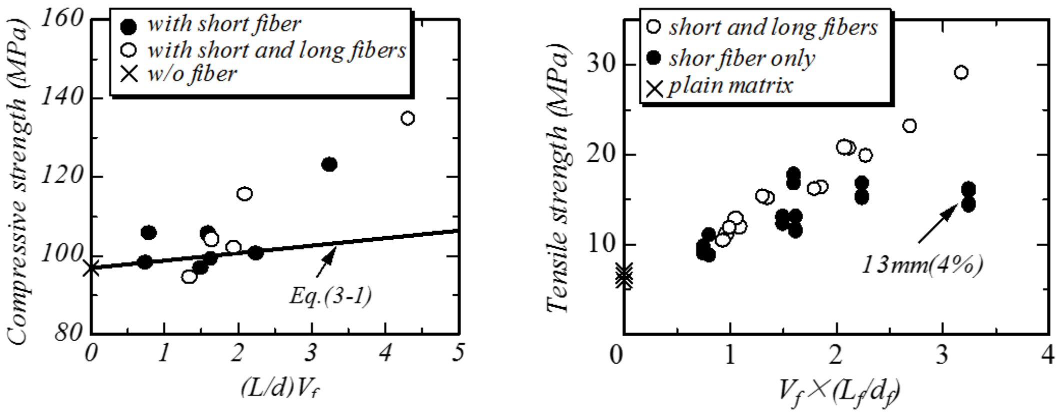


Figure 7 – Comparing the fibre factor of steel fibres with:
a) compressive strength and b) uni–axial tensile strength [15].

However, short fibres only mixtures at higher dosages deviated from the linear trend of mixtures of fibres reflecting a decreased efficiency caused by a pronounced effect of a high fibre number. Because of size differences a relative higher fibre dosage can be added for fibre combinations.

Performance of fibre reinforced concrete

In order to study the effect of the flow on fibre orientation and the relation between fibre orientation and structural performance two tunnel segments were cast with SCC containing steel fibres [16]. The fibre type (fibre dosage: 60 kg/m^3) was a parameter in this study (Dramix 45/30 BN/ L_f=30 mm or Dramix 80/60 BN/L_f=60 mm). Thirty hardened cylinders were drilled from each tunnel segment. Deformation-controlled splitting tensile tests were executed on fifteen cylinders. Fifteen additional cylinders were drilled point-symmetric to the point of casting (cast in the middle of the segment) to take X-ray photographs of concrete slices (d = 18 mm) and to determine the orientation of the fibres (by manually counting). Figure 8 shows the results of the splitting tensile tests of vertical cylinders in the middle at the end of the mould (the position with the largest difference in fibre orientation in two directions). The fibres were oriented mainly parallel to the walls of the tunnel segments, where the largest stresses are expected during positioning of the tunnel elements. In spite of considerable differences in two directions the post-cracking strength in both cases was higher than 80% of the first-cracking strength (Figure 8b). Reasons for an important contribution of the fibres in the weaker direction are:

- Usually, fibres are not aligned only in one direction (uni-axial) - a distribution of fibre orientation was observed by Laranjeira [17].
- The results of Figure 9 show that adding twice the amount of fibres did not improve the performance by a factor of two. It should be taken into account that a considerable contribution of the fibres also requires sufficient fibres in a cross-section.

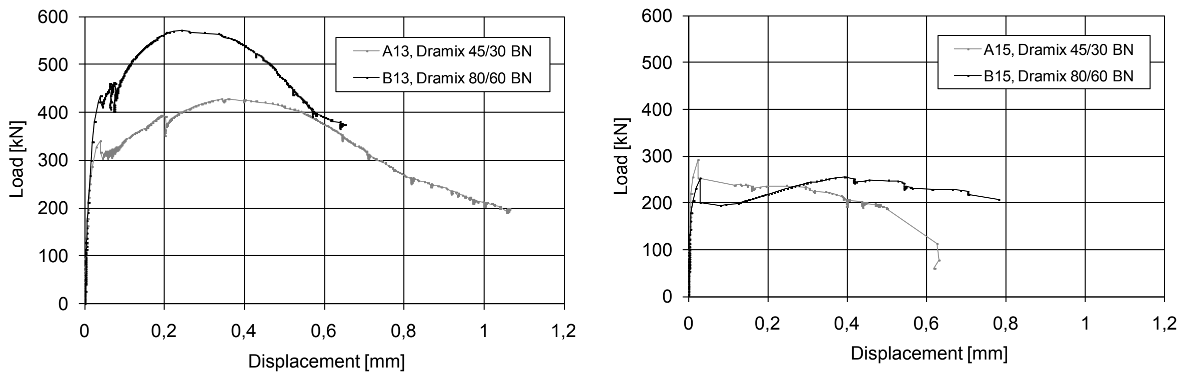


Figure 8 – Splitting tensile tests on cylinders drilled from two tunnel segments:
a) crack in flow direction and b) crack perpendicular to flow direction [16].

Six thin plates (dimensions: 600·600·15 mm^3) were produced as panels for a double floor system [18]. The thickness of the plates in this study was fixed to 15 mm. With a plate thickness of 15 mm and rebars having a diameter of 4-5 mm only a thin concrete cover (+/- 5 mm) can be realized. Instead the panels were reinforced with steel fibres only. The dosage of steel fibres (straight, L_f=13 mm, d_f=0.20 mm) was varied (0; 0.99 and 1.97 Vol.-%); two plates were produced with each mixture (Figure 9b shows Plate 3 after testing). The compressive strengths were 148, 145 and 164 MPa 56 days after casting for 0, 50 and 100% of the fibre dosage, respectively. The fibres (Panels 3-6) significantly increased the maximum load and ductility of the plates (Figure 9a) compared to the reference plates (Plates 1-2). The load-increase (average of two results) was 393% relative to the maximum load obtained with the reference plates (without fibres) with 50% of the maximum fibre dosage and 507% with 100% of the maximum fibre dosage, respectively. By doubling the fibre dosage the maximum flexural strength was increased by only a moderate 29% relative to plates containing half the fibre dosage, which reflects the potential for the optimization of fibre reinforced concrete resulting in a trade-off of panel geometry and fibre contribution and costs.

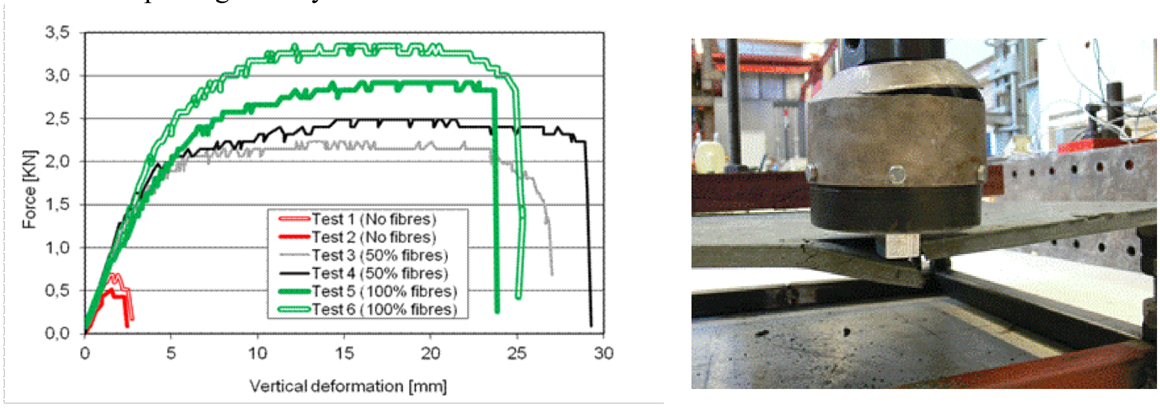


Figure 9 – Improvement of the load bearing capacity of thin plates (d=15 mm) due to steel fibres (L_f=13 mm) a) load–deformation curves and b) local failure of Plate 3 with 77.5 kg/m³ fibres [18].

Proven K-factor concept

In MC2010, the K-factor concept was implemented. The K-factor was introduced in the first edition of the French UHPC-recommendation in 2002 and takes into account different influence parameters (concerning production and workability) and allows to transfer results of small scale specimens to structures made with UHPFRC. The K-factor validates design assumptions as a result of tests on parts of full-scale test elements. Simon et al. [19] describe the K-factor concept and discuss the robustness and reliability with case-studies used as a reference. In several cases, K-factors lower than 1 were found, which is considered the minimum design value. In such cases, the performance of a part of a full-scale element was better than the experimental results of laboratory specimens. In case of the Pont Du Diable footbridge the highest obtained local K-factor was 2.12. The French UHPC-recommendation proposes as a first design approximation 1.75 for local effects and 1.25 for global effects. Such factors still have to be determined for a specific structure and validated for fibre reinforced concrete types other than UHPFRC.

STRUCTURAL APPLICATIONS WITH FLOWABLE CONCRETE

Fibres counteract the growth of cracks, hence realizing smaller crack widths at a given stress distribution and more crack surface can be activated. Fibres can make a significant contribution to the shear capacity of structural elements and they are able to supplement or replace rebars as shear reinforcement, which has benefits related to production efficiency and costs. Especially, thin elements are an interesting application for which the production and placement of reinforcement meshes is not possible or difficult to realize. MC2010 proposes the following equation to calculate the shear capacity, this provision is discussed in more detail by Di Prisco et al. [20].

$$V_{Rd,F} = \left(\frac{0.18}{\gamma_c} \cdot k \cdot \left[100 \cdot \rho_1 \cdot \left(1 + 7.5 \cdot \frac{f_{Ftuk}}{f_{ctk}} \right) \cdot f_{ck} \right]^{1/3} + 0.15 \cdot \sigma_{cp} \right) \cdot b_w \cdot d \qquad \text{[Equation 1]}$$

with

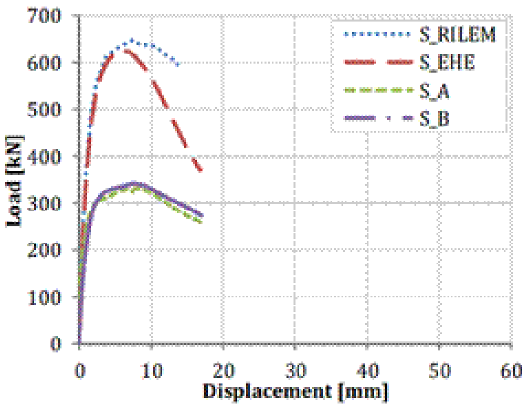
- γ_c is the partial safety factor for the concrete without fibres (1.5);
- k is a factor taking the effect of the depth of the cross-section into account
  $k = 1 + \sqrt{\frac{200}{d}} \leq 2.0$ with d in mm (as defined before);
- ρ_1 the longitudinal reinforcement ratio: $\rho_1 = \frac{A_{sl}}{b_w \cdot d}$;

- f_{Ftuk} is the characteristic value of the ultimate flexural residual tensile strength of the fibre reinforced concrete by considering wu=1.5 mm [MPa];
- f_{ctk} is the characteristic value of the tensile strength for the concrete matrix [MPa];
- σ_{cp} is the average stress acting on the cross-section due to an axial force (loading or prestressing action);

b_w is the smallest width of the cross-section in the tensile area.

Parmentier et al. [21] tested 28 reinforced concrete beams without stirrups and varied the type and dosage of the fibres (polypropylene fibres: 4.5 kg/m^3 and steel fibres: 20 or 40 kg/m^3) and the span-to-depth ratio (0.5, 1.5 and 2.5). All specimens showed a drop of the load after the peak in flexural tests (3-point bending test with notch) was reached. The increase in shear strength was in the range of 48-82%. The experimental results were compared with the predicted shear contribution for FRC. The shear strength was underestimated in all cases. In average the underestimation was 30% and the model was less accurate for span a/d-ratios of 2.5. Cauberg et al. [22] tested 45 beams in order to determine the shear capacity of UHPFRC. The dimensions of the beams were 2.3·0.16·0.25 m^3 (L/W/H). Reinforcement was placed for flexural strength and as shear reinforcement. The compressive strengths of the three applied mixtures were in the range of 130-160 MPa. They found that the equation proposed by MC2010 (Equation 1) significantly underestimated the shear capacity of beams with UHPFRC; the largest difference was observed for an a/d-ratio of 3. These results suggest that the provision of MC2010 is on the safe side, underestimating the experimental results more or less. Especially for high strength and ultra-high strength concretes (compressive strength in excess of C100/115) more results and comparisons are required to validate the application of Model Code provisions for all concrete types including mixtures of strengths higher than C120/140.

Blanco Álvarez [6] and Pujadas Álvarez [23] tested floors with steel or plastic fibres being the only reinforcement. Three geometries of the floors were considered: a square floor (3·3·0.2 m^3) and two floors with a shorter side (3·2·0.2 m^3 and 3·1·0.2 m^3). Concrete was cast in the middle and it was distributed with slight external vibration in the element. The floors were tested with a single load acting in the centre of the floors. With the concretes also prisms were cast, which were tested in bending. From the bending tests characteristic values were determined for the tensile behaviour. The constitutive behaviour was then used as an input for finite element calculations. As a result, a significant overestimation of the capacity of all floors was obtained; the difference decreased at increasing width of the floors. Figure 10 compares FEM-simulations and experimental results for floors with surfaces of 3m·1m (left) and 3m·3m (right).



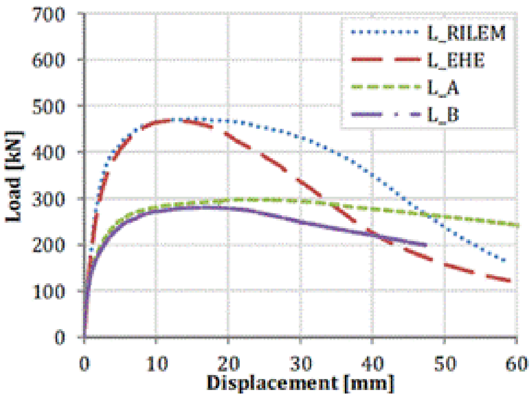


Figure 10 – Comparison of experimental results of floors compared with simulations (tensile behaviour according to RILEM- or Spanish EHE-codes) for a) the smallest floor (surface: 3m·1m) and b) the largest floor (surface: 3m·3m) [6].

The differences between simulations and experimental results were explained by differences in specimen geometry and structural size favouring the fibre orientation in small specimens. The performance of the larger floors was relatively better compared to shorter floors, since the fibres orient in a free-flow situation perpendicular to the flow direction which was better realized with a width of 3 m compared to 1 or 2 m. With the inductive method an indication of the fibre orientation was obtained (Fig. 11). Blanco Álvarez [6] divided the floors in three zones of characteristic fibre orientation (casting area, wall and intermediate area) and hereby explained the differences in load-bearing capacity of the floors. Size factors were proposed to transfer results of small prisms to the performance of large floors.

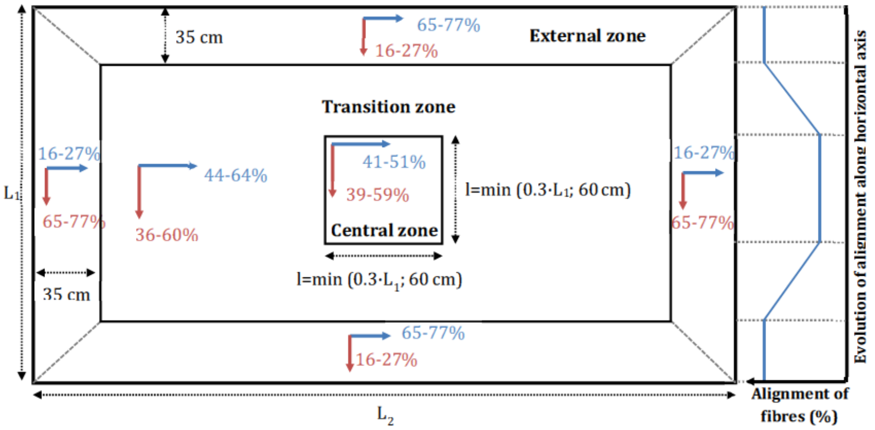


Figure 11 - Division of a slab in areas depending on the fibre orientation [6].

CONCLUSIONS

This paper discussed flowable concrete from the perspective of fib Task Group 8.8. For the purpose of underlining important conclusions case-studies were selected. The main conclusions are:

- Model Code 2010, the French UHPFRC-recommendation and the Japanese HPFRCC-guideline already cover a wide range of concretes concerning compressive strength and fibre content. fib TG 8.8 collects data and discusses the applicability of such provisions for flowable concrete.
- Local phenomena can have a pronounced effect on concrete (not only on self-compacting concrete but also on vibrated concrete).
- The development of meaningful and practical tests is important for the application of flowable concrete; more knowledge is required about how to translate such results in structural performance.
- Innovative concrete solutions and structures can be developed with tailor-made flowable concrete.

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