

Bulletin 79



Fibre-reinforced concrete: From design to structural applications FRC 2014:ACI-*fib* International Workshop



ACI SP-310
ACI-*fib* workshop proceedings
AVA Documents
Document Sharing Hub

fib 79 CEB • FIP Bulletin

Fibre-reinforced concrete: From design to structural applications



American Concrete Institute
Always advancing

Technical report

Proceedings of a workshop held in Montreal, Canada,
on 24 and 25 July 2014

Edited by Bruno Massicotte, Jean-Philippe Charron,
Giovanni Plizzari and Barzin Mobasher

May 2016

Subject to priorities defined by the technical council and the presidium, the results of the *fib*'s work in commissions and task groups are published in a continuously numbered series of technical publications called *bulletins*. The following categories are used:

Category:

Technical report
State-of-the-art report
Manual / Guide to good practice / Recommendation
Model code

Draft to be approved by:

Task group and chairpersons of the commission
Commission
Technical council
General assembly

Any publication not having met the above requirements will be clearly identified as a preliminary draft.

fib Bulletin 79 is published as a technical report and is a collection of contributions to a workshop that was co-sponsored by the *fib* and the American Concrete Institute (ACI). The authors have presented their individual views. Although these contributions have not been discussed in any of the *fib*'s working bodies, the subject matter is highly topical and believed to be of general interest to members of the *fib*.

This bulletin is also published as an ACI Symposium Publication, ACI SP-310.

Cover images:

Front cover: Illustration based on a photograph of UHPFRC precast segments for the 100-metre-span box girder Batu 6 Bridge, crossing Sungai Perak, Malaysia. Photograph courtesy of Stephen Foster, University of New South Wales, Australia.

Back cover: Illustration based on a photograph of a full-scale 5-metre-long prototype of the 43.5-metre-span UHPFRC truss footbridge over the Ovejas ravine in Alicante, Spain. Photograph courtesy of Pedro Serna, Universitat Politècnica de Valencia, Spain.

© Fédération internationale du béton (*fib*) and American Concrete Institute (ACI), 2016

Although the International Federation for Structural Concrete / Fédération internationale du béton (*fib*) and the American Concrete Institute (ACI) do their best to ensure that all the information presented in this publication is accurate, no liability or responsibility of any kind, including liability for negligence, is accepted in this respect by these organizations, their members, employees or agents.

All rights reserved. No part of this publication may be reproduced, modified, translated, stored in a retrieval system or transmitted in any form or by any means - electronically, mechanically, through photocopying, recording or otherwise - without prior written permission from the *fib*.

ISSN 1562-3610

ISBN 978-2-88394-119-9

Printed by DCC Document Competence Center Siegmars K stl e.K., Germany

Preface

The FRC-2014 Workshop, *Fibre-reinforced concrete: From design to structural applications*, was the first ever ACI-*fib* joint technical event. The workshop, held at Polytechnique Montreal, Canada, on July 24 and 25, 2014, was attended by 116 participants from 25 countries and four continents.

The first international FRC workshop was held in Bergamo, Italy, in 2004. At that time, the lack of specific building codes and standards was identified as the main inhibitor to the application of this technology in engineering practice. Ten years after Bergamo, many of the objectives identified at that time have been achieved. The use of fibre-reinforced concrete (FRC) for designing structural members in bending and shear has recently been addressed in the *fib* Model Code for Concrete Structures 2010. Steel-fibre-reinforced concrete (SFRC) has also been used structurally in several building and bridge projects in Europe and North America. SFRC has been widely used in segmental tunnel linings all over the world. Members of ACI 544 and *fib* TG 4.1 have been involved in writing code-based specifications for the design of FRC structural members.

Although fibres have been used by the construction industry for several decades, their use in structural applications is still very modest if one considers the gigantic potential of concrete structures around the world and the benefits expected of their mechanical behaviour and durability. However, recent technological developments and large scale applications have demonstrated that FRC has reached a level of maturity such that these innovative materials can be used by engineers with confidence. From that perspective, the aim of the FRC 2014 workshop was to provide the state-of-the-art on the recent progress attained in terms of specifications and actual applications. Presentations covered several design guidelines adopted worldwide illustrating the progress made in the last ten years, and also a wide spectrum of FRC applications such as beams, elevated floors, tunnel linings, slabs, pavements, precast elements, bridge elements, and many others.

More than fifty papers were presented at the workshop, from which 44 were selected for this joint ACI-*fib* publication. The papers are organised into six themes:

- design guidelines and specifications,
- material properties for design,
- behaviour and design of beams and columns,
- behaviour and design of slabs and other structures,
- behaviour and design of foundations and underground components, and finally,
- applications in structure and underground construction projects.

The papers cover a wide range of applications and illustrate the maturity of FRC as the choice material for improving the serviceability, sustainability, and performance of concrete structures. The workshop chairs would like to express their sincere recognition to all authors and reviewers who contributed to the quality of the document. Special thanks to both ACI and *fib* officers and staff who supported the organisation of the workshop, editorial support, and dissemination of the workshop proceeding as an ACI Symposium Publication and an *fib* Bulletin.

While significant progress has been made in the introduction of FRC in codes and structures, the current accomplishment should be viewed as the beginning, and significant follow up work is still needed. Indeed, introducing new technologies and new materials in structural applications brings technical and scientific challenges and responsibilities. The necessity to achieve the objectives set worldwide for sustainable development requires that 21st century concrete structures meet higher performances than those of the previous one, a role that FRC can definitely help achieve. It is the responsibility of all actors to move forward in that direction.

The recently published codes and design guidelines, available worldwide, constitute a first step into the implementation of FRC in the construction industry. However, before the structural use of FRC becomes a common practice, several benchmarks need to be accomplished. The numerous factors that still inhibit the use of FRC in structural applications should be viewed as challenges that could only be solved through a joint effort of all key players. Professor Sidney Mindess, a pioneer of FRC research, indicated in his opening speech at the Montreal Workshop four challenges to increase the structural use of FRC: education and training, performance specifications, more appropriate testing methods, and comprehensive research programs focused on the combined use of FRC and continuous reinforcement. Without claiming to provide an exhaustive list of actions, the following presents some challenges that need to be addressed by the scientific community, fibre producers, structural design community, construction industry, and stakeholders to achieve the objective of building more durable concrete structures. These challenges are inspired by Professor Mindess' remarks.

The scientific community should contribute to the elimination of artificial divisions between different types of FRC based on the compressive strength and type of fibres. They should put the emphasis on the benefits brought by fibres on the performance of concrete structures and present FRC as a continuum of materials with different characteristics and performances. Approaching research on FRC more holistically with the emphasis on applications is essential for the sound development of the scientific knowledge.

Fibre producers would certainly be the first ones to benefit from a wider use of fibre concrete. Considering that about 1% of the concrete used worldwide contains fibres, they should combine their efforts for developing new applications and expanding the spectrum of the conventional uses, rather than working against each other for the same market. Addressing the replacement of alternative reinforcement solutions should evolve toward adopting a strategy based on the vision that fibres should be used along with other types of reinforcement to make better constructions at both service and ultimate limit states.

The structural design community has always adopted conservatism that is justified in many instances for safety and professional responsibility. The evidence of enhanced performances of FRC structures and the need to build more durable and safer structures only justifies a more extensive use of FRC. Being at the decision central point, designers should be more proactive in proposing the implementation of FRC in structures. They should contribute to the writing of design guidelines, they should ask to get appropriate training, and they should promote the improvement of the expertise level of the engineering profession.

Being driven by the necessity of profit, the construction industry has always been resistant to changes unless motivated by economic advantages. Problems associated with the use of FRC, especially at high dosages, have often and justifiably discouraged their broader use. Today's technological knowhow and availability of products are such that past technical

problems have been overcome. Changing traditional ways of building with FRC will need some effort and modification of the current practice. However, members of the construction industry with the vision of tomorrow's concrete structures will make the appropriate changes because survival often requires evolution. A better and more modern image of the construction industry would certainly be beneficial to all.

Stakeholders with long-term vision will implement the needed change, as clear evidence of better, safer, and more durable structures with FRC is needed. Combined with the maturity of the scientific knowledge on FRC, stakeholders can now require a wider use of FRC. When only short-term economic considerations prevail, FRC is not always competitive. However, when better service performances, higher longevity and enhanced quality become important issues, FRC utilisation often becomes inevitable. Therefore, those who decide the quality of concrete should be able to defend their choice to those they represent. The onus is on them to justify not using FRC considering all benefits brought by their appropriate use in structures.

Not all challenges have been discussed here and several technical and scientific issues still have to be resolved. Adding fibres into a concrete mix is not magical and the challenges remain high. Despite the obstacles that need to be crossed and the long journey ahead, the path appears more clearly. It requires more research, open minds, visions and close collaboration between all actors. Forums such as the FRC workshops are essential events that bring together participants of various technical geographical origin. They are one component of the collaborative effort that is needed to achieve the objective of building better structures.

FRC is a remarkable material, and so far we have only scratched the surface of the contributions it can make to structural concrete design. Although fibres themselves are relatively expensive, they lead to real economic benefits in the design of concrete structures, and can expand the range of structures that can be constructed using concrete. It is hoped that some of the suggestions presented above can lead to the more rapid introduction of this material into everyday engineering practice.

Bruno Massicotte, Jean-Philippe Charron, Giovanni Plizzari and Barzin Mobasher

Contents

Preface	iii
Design guidelines and specifications	
Structural design according to <i>fib</i> MC 2010: Comparison between RC and FRC elements <i>Marco di Prisco, Giovanni Plizzari, Lucie Vandewalle</i>	1
Design based approaches for fibre-reinforced concrete: An overview of ACI committee 544 activities <i>Barzin Mobasher</i>	17
FRC design according to the draft Australian bridge code <i>Stephen Foster</i>	29
An introduction to the Chinese guideline for fibre-reinforced concrete structures <i>Christopher K. Y. Leung</i>	41
FRCC: Design and application in Japan <i>Yuichi Uchida, Minoru Kunieda, Keitetsu Rokugo</i>	51
French recommendations and feedback on experience with ultra-high-performance fibre-reinforced concrete (UHPFRC) <i>Jacques Resplendino</i>	61
Steel-fibre-reinforced concrete (SFRC) in fire: Normative and pre-normative requirements and code-type regulations <i>Frank Dehn, Annemarie Herrmann</i>	75
Material properties for design	
Translation of test results of small specimens of flowable fibre concrete to structural behaviour: A discussion paper of <i>fib</i> Task Group 4.3 <i>Steffen Grünewald, Luca Bartoli, Liberato Ferrara, Terje Kanstad, Frank Dehn</i>	81
Fibre-reinforced cementitious composites with adapted rheology: From state-of-the-art knowledge towards new boundaries for structural concrete applications <i>Liberato Ferrara</i>	91
Study of rheological and mechanical performance of ultra-high-performance glass concrete <i>Nancy Soliman, Arezki Tagnit-Hamou</i>	103
Feasibility of using recycled steel fibres to enhance the behaviour of recycled aggregate concrete <i>Khaleel H. Younis, Kypros Pilakoutas, Maurizio Guadagnini, Harris Angelakopoulos</i>	113

Effect of steel fibres on the tensile behaviour of self-consolidating reinforced concrete blocks	123
<i>Romildo Dias Toledo Filho, Ederli Marangon, Flávio de Andrade Silva, Barzin Mobasher</i>	
Fracture behaviour of polyolefin fibre-reinforced self-compacting concrete	131
<i>Marcos G. Alberti, Alejandro Enfedaque, Jaime C. Gálvez</i>	
Modelling early age drying in fibre-reinforced concretes	141
<i>Tara Rahmani, Mehdi Bakhshi, Barzin Mobasher, Mohammad Shekarchi</i>	
Behaviour and design of beams and columns	
Shear design of full-scale prestressed SFRC girders	151
<i>Tim Soetens, Stijn Matthys</i>	
Numerical modelling of large scale steel-fibre-reinforced reinforced concrete beams failing in shear	161
<i>Ali Amin, Stephen J. Foster</i>	
Structural applicability of polypropylene fibres: Deep and wide-shallow beams subjected to shear	171
<i>Antonio Conforti, Andrea Tinini, Fausto Minelli, Giovanni Plizzari, Sandro Moro</i>	
The effect of fibres in UHPFRC beams with longitudinal steel reinforcement	181
<i>Norbert Randl, Tamás Mészöly</i>	
Experimental and numerical study on the use of high-strength and ultra-high-performance fibre-reinforced concrete in columns	193
<i>Dario Redaelli, Ana Spasojevic, Aurelio Muttoni</i>	
Experimental and analytical behaviour of RC members strengthened by means of a high-performance jacket	203
<i>Serena Mostosi, Consuelo Beschi, Alberto Meda, Paolo Riva</i>	
Behaviour and design of slabs and other structures	
The behaviour of SFRC flat slabs: The Limelette full-scale experiments to support design model codes	213
<i>Benoit Parmentier, Petra Van Itterbeeck, Audrey Skowron</i>	
Precast fibre-reinforced self-compacting concrete slabs	223
<i>Luca Facconi, Fausto Minelli, Giovanni Plizzari, Andrea Pasetto</i>	
Precast plates made with lightweight fibre-reinforced concrete	239
<i>Alessandro P. Fantilli, Andrea Gorino, Bernardino Chiaia</i>	
Mechanical behaviour of slabs made of strain-hardening cement-based composite and steel reinforcement subject to uniaxial tensile loading	249
<i>Eric Mündecke, Viktor Mechtcherine</i>	

Innovative precast HPFRC barriers for bridges <i>Jean-Philippe Charron, Matthew Namy, François Duchesneau, Bruno Massicotte</i>	259
Effectiveness of fibres for structural elements in case of fire <i>György L. Balázs, Éva Lubláy, Olivér A. Czoboly</i>	269
Steel-fibre reinforcement for large hydraulic concrete structures: Numerical investigations on a semi-spiral case <i>Mahdi Ben Ftima, Bruno Massicotte, Sébastien Mousseau</i>	279
Numerical models for designing steel-fibre-reinforced concrete structures: Why and which ones? <i>Pierre Rossi, Jean-Louis Tailhan, Dominic Daviau-Desnoyers</i>	289
Load-bearing capacities of SFRC elements accounting for tension stiffening with modified moment-curvature relations <i>Peter Heek, Peter Mark</i>	301
Behaviour and design of foundations and underground components	
Design of glass-fibre-reinforced concrete floors according to the <i>fib</i> Model Code 2010 <i>Bryan Barragán, Luca Facconi, Olivier Laurence, Giovanni Plizzari</i>	311
A comprehensive methodology for the design of fibre-reinforced concrete pavements <i>Sunitha K. Nayar, Ravindra Gettu</i>	321
Experimental and numerical study on the load-bearing behaviour of steel-fibre-reinforced concrete for precast tunnel lining segments under concentrated loads <i>Rolf Breitenbücher, Günther Meschke, Fanbing Song, Michael Hofmann, Yijian Zhan</i>	331
Developments in design for fibre-reinforced concrete tunnel segments <i>Mehdi Bakhshi, Varya Nasri</i>	341
Optimization of input parameters for material modelling of fibre-reinforced concrete and application to the numerical simulation of tunnel lining <i>Tereza Sajdlová, Radomír Pukl</i>	353
Steel-fibre reinforcement for precast lining in tunnels with different diameters <i>Alberto Meda, Zila Rinaldi</i>	363
Numerical simulation of steel-fibre-reinforced concrete pipes using constitutive equations based on the Barcelona test <i>Renata Monte, Albert de la Fuente, Antonio D. de Figueiredo, Antonio Aguado</i>	373
Optimum fibre content for precast steel-fibre-reinforced concrete pipes <i>Nedal Mohamed, Ahmed Soliman, Moncef Nehdi</i>	383

Applications in structure and underground construction projects

FRC structural applications according to the <i>fib</i> Model Code 2010: A unified approach <i>Marco di Prisco, Matteo Colombo, Pamela Bonalumi, Carlo Beltrami</i>	393
Steel-fibre-reinforced concrete elevated suspended slabs: Design cases in Europe and the USA <i>Xavier Destrée</i>	409
Design and construction of SFRC bridge decks: Building on past experiences and recent developments <i>Bruno Massicotte, Léa Faggio, Nicola Cordonì, Ali Nour, David Conciatori</i>	419
Footbridge over the Ovejas ravine in Alicante: An economical alternative made only of ultra-high-performance fibre-reinforced concrete (UHPFRC) <i>Pedro Serna, Juan Ángel López, Esteban Camacho, Hugo Coll, Juan Navarro-Gregori</i>	435
Examples of bridge, tunnel lining and foundation design with steel-fibre-reinforced concrete <i>Agnieszka Winkler, Carola Edvardsen, Thomas Kasper</i>	451
Earthquake-resistant fibre-reinforced concrete coupling beams without diagonal bars <i>G.J. Parra-Montesinos, J. K. Wight, C. Kopczynski, R.D. Lequesne, M. Setkit, A. Conforti, J. Ferzli</i>	461
Design of FRC tunnel segments considering the ductility requirements of the <i>fib</i> Model Code 2010: Application to the Barcelona Metro Line 9 <i>Albert de la Fuente, Liao Lin, Sergio Cavalaro, Antonio Aguado</i>	471

Structural design according to *fib* MC 2010: Comparison between RC and FRC elements

Marco di Prisco¹, Giovanni Plizzari², Lucie Vandewalle³

¹ : Politecnico di Milano, Italy

² : University of Brescia, Italy

³ : KU Leuven, Belgium

Abstract

After several decades of research work and some years of pioneer applications, fibre-reinforced concrete (FRC) is nowadays a material ready for the world community, also considering that design rules are already available in several countries and the *fib* Model Code 2010 includes specific sections for design of FRC elements. FRC can be a suitable solution, especially for statically indeterminate structures, where stress redistribution occurs. In addition to the structural bearing capacity, FRC is particularly useful for better controlling crack opening in service conditions, which has a particular influence on structural durability, especially in aggressive environments. Furthermore, structural robustness is nowadays a major concern among structural engineers. Also in this perspective, FRC could improve structural behaviour since it provides structural resistance both in compression and in tension in all the regions of the structural element. In the present paper, the design procedure is applied to some structural elements where FRC may represent a suitable material for structural behaviour. Beside structural strength, crack opening in service conditions is determined and comparison in terms of total amount of reinforcement (fibres + rebars) is presented.

Keywords

Conceptual design, FRC elements, wide-shallow beams, suspended slab.

1 Introduction

After several decades of research work (ACI, 1996; Rossi and Chanvillard, 2000; di Prisco et. al, 2004; Reinhardt and Naaman, 2007; Gettu, 2008; Barros, 2012) and some years of pioneer applications, fibre-reinforced concrete (FRC) is now ready for the world community, also considering that design rules are now available in several Countries and that the new *fib* Model Code 2010 (2012) includes specific sections for design of FRC elements (di Prisco et al., 2009). It is now clear that FRC is not simply the addition of fibres to concrete but it is composite whose properties must be specified by engineers and controlled at the constructions site. Fibres can be made of different materials, such as steel, polypropylene, polyester, polyvinyl alcohol or glass and cocktails of fibres can be properly considered for enhancing material performance. The important aspect for structural design is the material performance which can be determined with standard tests. The peculiar property of FRC is the residual post-cracking strength, often defined as toughness in the following, which makes FRC very promising in elements subjected to diffused stresses where its enhanced residual post-cracking strength can better resist tensile

stresses diffused in the structural element. After cracking, depending on the properties of the concrete matrix and of the fibres (type, material, content), FRC can have a hardening behaviour or a softening behaviour (either in bending or in the uniaxial tension); the latter being significantly different than reinforced concrete (RC), FRC can be a suitable solution for statically indeterminate structures where stress redistribution occurs. For this reason, MC 2010 (2012) introduced special ductility requirements for FRC structures that can be satisfied when the structure has a high degree of redundancy; otherwise, some rebars should be introduced along lines where stresses tend to localize. Another significant aspect of FRC concerns the fibre orientation. In fact, there are no doubts that, in general, a 3D distribution of fibres in the structure is the desired solution; however, there are particular elements (as thin ones) where fibres tend to orient (depending on their length with respect to the element thickness) along the main element plane. These elements can take advantage from fibre orientation since they may be aligned to the main structural stresses. In order to better evaluate the material properties, special “structural specimens”, that better represent the real structure, may be adopted (MC 2010, 2012). In addition to the structural bearing capacity, FRC is particularly useful to better control crack opening in service conditions, which has a particular influence on structural durability, especially in aggressive environments (Vasanelli et al., 2013). Last but not least, structural robustness is nowadays a major concern among structural engineers; also in this perspective, FRC could enhance structural behaviour since it provides structural resistance, both in compression and in tension, in all the volume of the structural element.

In the present paper, the design procedure is applied to some structural elements where FRC can represent an optimal material for structural behaviour. Besides structural strength, crack opening in service conditions is determined and comparison in terms of total amount of reinforcement (fibres + rebars) is presented.

2 Materials

The design rules reported herein (for reinforced concrete – RC – and fibre-reinforced concrete – FRC) are present in the Model Code 2010 (2012). Therefore, the safety factors of materials adopted are $\gamma_c = 1.5$ for concrete in compression, $\gamma_F = 1.5$ for FRC in tension, and $\gamma_s = 1.15$ for steel rebars.

Moreover, fundamental and characteristic load combinations were considered at Ultimate Limit State (ULS) and Serviceability Limit State (SLS), respectively. The FRC used to exemplify the comparison in the design has a class 3c (Section 5.6.3 of MC 2010) which, as an example, can be obtained quite easily with a concrete C30/37 and an amount of steel fibres ranging between 30 and 40 kg/m³.

3 Case study 1: Wide-shallow beams

A wide-shallow beam (WSB), 80 cm wide and 30 cm deep, is used as the central beam in a lightweight ribbed one-way RC slab of a residential building. It is supported by three columns, leading to a two-span continuous beam (with both net spans of 500 cm).

Considering a gross cover (*c*) of 4 cm, the WSB effective depth (*d*) results 26 cm. The dead load and the live load are 40 kN/m and 10 kN/m, respectively. The WSB is made of concrete C30/37 and by rebar B450 (characteristic yielding strength of 450 MPa).

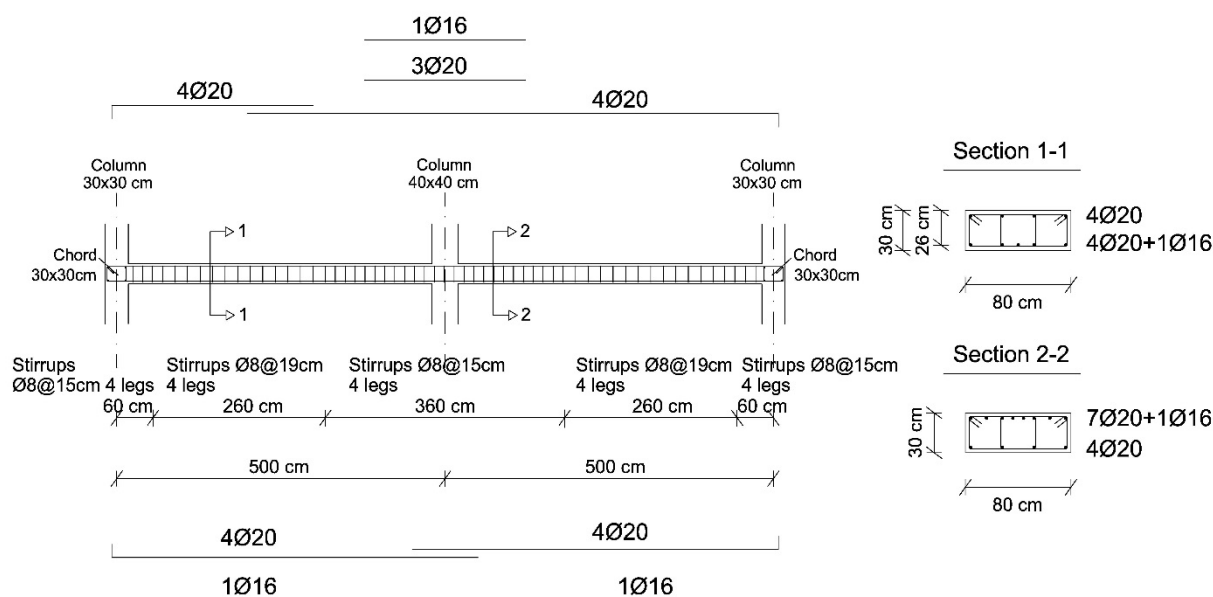


Figure 1: Geometry and reinforcement details of RC WSB 800 mm wide and 300 mm high.

In order to compare the RC and FRC solutions, firstly the WSB is totally designed in reinforced concrete (with both longitudinal and transverse reinforcement). The reinforcement details of the beam are reported in Figure 1. Rebars Ø16 and Ø20 are used as longitudinal reinforcement, giving a reinforcement ratio of about 1.15%. Rebars Ø20 are also used in the compressive zone to hold the stirrups in the correct position. Concerning transverse reinforcement, the design is done considering Level I approximation of Model Code 2010 (Eqs. 7.3-11, 7.3-17, 7.3-19, 7.3-22, 7.3-29 and 7.3-36), with inclination of the compressive stress field of 30°. Therefore, stirrups Ø8@19cm with 4 legs are used as minimum transverse reinforcement (according also to Section 7.13.5 of Model Code 2010) and stirrups Ø8@15cm with 4 legs as equilibrium shear reinforcement. Concerning SLS, the crack width calculation is also based on MC 2010 (Eqs 7.6-3 and 7.6-4); these equations lead to a maximum crack spacing of 93 mm and to a maximum crack width of about 0.21 mm.

Recent studies confirm the structural applicability of fibres in WSBs (Conforti et al., 2013). In fact fibres, even in small amounts, significantly improve the behaviour of WSBs both at SLS (higher post-cracking stiffness and crack width control) and ULS (shear and flexural strength). Moreover, the effectiveness of fibres as shear reinforcement was also confirmed in other beam typologies, such as deep beams (Dinh et al., 2010; Minelli et al., 2014), prestressed beams (Cuenca et al., 2013) and hollow core slabs (Cuenca et al., 2013). Therefore, the same WSB shown in Figure 1 is designed with FRC, considering a FRC class 3c. The reinforcement details of the FRC WSB are reported in Figure 2. Fibres contribution is considered in the evaluation of design bending resistance (Section 5.6.4 of MC 2010), leading to a reduction of the longitudinal reinforcement, as compared to the RC solution (Figure 1). It can be noticed that fibres increase the design bending resistance of about 10-15%. However, fibres are mainly effective as shear reinforcement, both in terms of minimum and equilibrium transverse reinforcement. In fact, the FRC adopted (C30/37 – FRC 3c) fulfils Equation 7.7-14 of MC 2010 and, therefore, conventional minimum shear reinforcement is completely substituted by fibres. Around the columns, the design value of shear ($V_{Ed}=210$ kN at central column, $V_{Ed}=140$ kN at lateral columns) exceeds the design shear contribution of concrete ($V_{Rd,c}=95$ kN, evaluated by

Equations 7.3-17 and 7.3-19 of MC 2010). By Equation 7.7-4 of MC 2010, the shear resistance of FRC WSB ($V_{Rd,F}$) can be evaluated. Since the design shear resistance ($V_{Rd,F}=250$ kN) is greater than the design value of shear V_{Ed} , even the equilibrium shear reinforcement can be provided by fibres only. A few stirrups ($\varnothing 8@20$ cm 4 legs) are placed around the central column (40×40 cm) for resisting the transverse bending (since WSB width is larger than the central column) and for keeping the longitudinal rebars in the correct position.

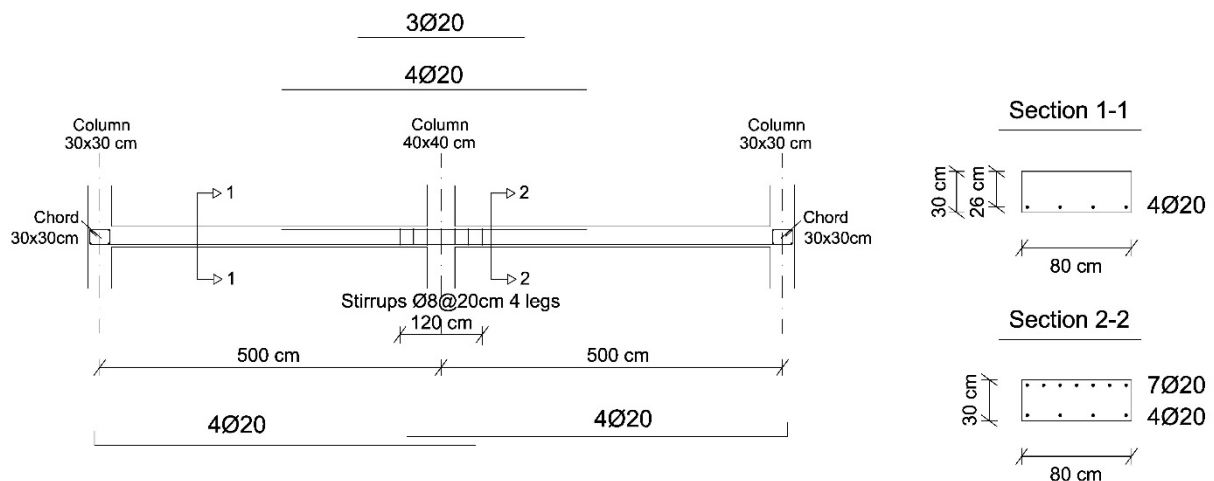


Figure 2: Geometry and reinforcement details of FRC WSB 800 mm wide and 300 mm high.

Moreover, crack spacing and crack width are calculated for FRC WSB as well (SLS). By using Equations 7.7-22, 7.7-23, 7.7-24 and assuming a linear model for fibres (Equation 5.6-5) of MC 2010, the crack spacing results in about 55 mm (40% smaller than RC) and the crack width is 0.14 mm (35% smaller than RC). The reduction of crack spacing and width due to the addition of fibres was experimentally proved by Conforti et al. (2013) in WSBs and by Minelli et al. (2014) in deep beams. It should be noticed that, as mentioned above, a C30/37 FRC classified as 3c could be made by using about 35 kg/m^3 of steel fibres 60 mm long with diameter of 0.75 mm. By considering the RC WSB of Figure 1 and the FRC WSB of Figure 2, the total steel content results in about 140 kg/m^3 and 115 kg/m^3 , respectively. Therefore, FRC solution leads to a reduction of about 20% of steel weight, in addition to the reduction of crack spacing and crack width, with the consequent durability enhancements.

4 Case study 2: Suspended slab

Two case studies regarding the design of slabs according to Model Code 2010 (2012) are presented in this Section. In particular, the first one is the reliability check of the equations to compute the ultimate bearing capacity of a pile-supported slab. The other is a detailed design example related to a suspended slab.

4.1 Comparison between the design bearing capacity and experimental results from a full-scale test on a pile supported FRC slab

The comparison refers to the full-scale test carried out by Hedebratt et al. (2013). The aim of the short-term study was to investigate the structural behaviour of pile supported slabs made of steel-fibre-reinforced concrete (SFRC) only. The elevated flat slab was supported by columns (piles), with a slab span of $L=3.0$ m and walls outside the perimeter of the outer column. The thickness (t) of the elevated slab was 130 mm. Eight slabs were tested (up to failure) with different materials and under different boundary conditions. In particular, the slabs were cast with two fibre contents with the aim of using both a softening and a hardening material in bending. Another difference concerns the presence of rebars. In the following, only test number 8 is considered with a clamped edge panel that undertook a flexural failure mode with a point load ($P_{\max}$) of 151.1 kN. For the evaluation of ULS, MC 2010 suggests the constitutive laws described in Section 5.6.4. In particular, a rigid-plastic model is adopted (assuming a FRC 3c) and the ultimate moment (for unit length) is equal to:

$$m_{uk}^{FRC} = 0.5 * t^2 * f_{Ftuk} = 0.5 * t^2 * f_{R3k} / 3 = 7.6 \text{ kN m/m} \quad (1)$$

Since the flat slab under investigation is clearly a structure able to significantly redistribute the applied loads, a factor K_{rd} , which takes into account this favourable effect, can be assumed (as suggested in Section 7.7.2 of MC 2010). The calibration of that parameter was done in accordance with the work carried out by Dozio (2008) and di Prisco et al. (2013, 2014) where K_{rd} was computed by a structural analysis that takes into account a random redistribution of the mechanical characteristics: for a plate, this value cannot exceed 1.4.

Due to the bending failure, the yield line method is used to predict the maximum load. For a clamped edge panel with a central point load, the failure pattern provides a maximum load:

$$P_{uk} = K_{Rd} * 12 * m_{uk}^{FRC} - 5/12 gl^2 = 1.4 * 12 * 7.6 - 11.7 \text{ kN m/m} = 116 \text{ kN} \quad (2)$$

The ratio between the experimental and the design bearing capacity of the plate is:

$$\Delta = P_{exp} / P_{uk} = (151 \text{ kN}) / (116 \text{ kN}) = 1.30 \quad (3)$$

which is acceptable also considering that K_{Rd} is probably a little underestimated and the design bearing capacity was determined with the characteristic value of the post-cracking strength f_{R3k} .

The slab investigated is designed as a FRC structure without conventional reinforcement; according to MC 2010 (Section 7.7.2), the ductility requirement in bending are satisfied if one the following conditions are verified:

$$\delta_u \geq 20 \delta_{SLS} \quad (4)$$

$$\delta_{peak} \geq 5 \delta_{SLS} \quad (5)$$

In order to determine the maximum displacement, one could refer to the rigid kinematic mechanism at failure, by assuming $w_{uk}=2.5$ mm; this crack opening was confirmed by the final crack pattern after the full-scale test, as reported in Hedebratt et al. (2013). Owing to the use of a rigid-plastic constitutive relationship, the computed displacement according to the kinematic model corresponds to the peak load (δ_{peak}).

The sectional rotation at failure is:

$$\theta = w_{uk}/(2 \cdot t) = 2.5 \text{ mm}/(2 \cdot 130 \text{ mm}) = 0.0096 \quad (6)$$

Therefore, the vertical displacement estimation related to the mechanism is:

$$\delta_{peak} = \theta \cdot L/2 = 0.0096 \cdot 3000/2 \text{ mm} = 14.4 \text{ mm} \quad (7)$$

From an elastic analysis of the slab subjected to a point load, the maximum displacement at service load (by assuming uncracked concrete and initial elastic Young's modulus) was equal to 2.15 mm. From this value it is possible to evaluate the ductility condition of Equation 5. The ratio between the peak and service displacement is equal to 6.7, which satisfies the requirements of MC 2010. In reality, the experimental displacement recorded during the full-scale test (that was carried out under load control) showed a higher vertical displacement, equal to 37.1 mm.

The deviation between the calculated response and the observed one may be caused by several reasons, including the membrane effect. It is worth noting that the assumption of 2.5 mm for the ultimate crack opening is validated also by the crack pattern experimentally observed, with crack widths ranging between 1.5 and 3 mm.

In order to consider all possible modes of failure, also the punching resistance of the slab was evaluated. According to MC 2010 (Eqs 7.7-18 and 7.7-19), the punching resistance is given by:

$$V_{Rd} = V_{Rd,F} + V_{Rd,s} \quad (8)$$

where

$$V_{Rd,F} = V_{Rd,c} + V_{Rd,f} \quad (9)$$

and $V_{Rd,f}$ is evaluated according to Equation 7.7-20 of MC 2010.

In particular, the design shear resistance attributed to the fibres may be calculated according to Equation 7.7-19 of MC 2010, by considering the 3c-class of FRC (di Prisco et al., 2013):

$$f_{Ftsk} = 0.37 \cdot f_{R1k} = 1.11 \text{ MPa} \quad (10)$$

$$w_u = 1.5 \text{ mm} \quad (11)$$

$$CMOD_3 = 2.5 \text{ mm} \quad (12)$$

The characteristic value of the ultimate residual tensile strength for FRC (Eq. 5.6-6) is:

$$f_{Ftuk} = f_{Ftsk} - w_u / CMOD_3 \cdot (f_{Ftsk} - 0.5 \cdot f_{R3k} + 0.2 \cdot f_{R1k}) = 0.89 \text{ MPa} \quad (13)$$

The shear resistant control perimeter (Eq. 7.3.5.2) is:

$$b_0 = 2 \cdot b^2 + \pi \cdot t = 2 \cdot (200 \text{ mm})^2 + \pi \cdot 130 \text{ mm} = 1208 \text{ mm} \quad (14)$$

Therefore, the fibre contribution is:

$$V_{Rk,f} = f_{Ftuk} \cdot b_0 \cdot t = 140 \text{ kN} \quad (15)$$

The characteristic shear resistance attributed to concrete may be taken as in Equation 7.3-61 of MC 2010, by considering the level I approximation. This results in:

$$V_{Rk,c} = k_{\psi} \cdot \sqrt{f_{ck}} \cdot b_0 \cdot t = 0.291 \cdot \sqrt{30} \cdot 1208 \cdot 130 = 250 \text{ kN} \quad (16)$$

Therefore, the total shear resistance is:

$$V_{Rk,F} = V_{Rk,c} + V_{Rk,f} = 391 \text{ kN} \gg P_{uk}/4 + gl^2/4 = (151.1/4 + 28.1/4) \text{ kN} = 44.8 \text{ kN} \quad (17)$$

It should be noticed that, in this verification, K_{rd} is not used because the failure considered is a local failure rather than a global structural failure, thus involving a negligible stress redistribution.

4.2 Design of flat slabs on a rectangular grid of columns

In order to compare the RC and FRC solution for a suspended slab, an example referring to the European concrete frame-building project is reported herein. This was a full-scale (seven-storey) RC building frame, constructed in the Building Research Establishment's Cardington laboratory (BRE). The slab had a thickness (t) of 25 cm and was supported by columns with a regular grid line (7.5 m x 7.5 m); the edge and corner columns were rectangular with a 25x40 cm cross section, while the central ones were 40x40 cm. A mean gross cover (c) equal to 3.5 cm for the top and bottom sides in each direction was considered. The characteristic dead load was equal to 7 kN/m², whereas the live load was 3.5 kN/m², resulting in $q_{sd} = 14.7 \text{ kN/m}^2$. The structural elements designed in the present paper are made of concrete C30/37 and rebars B500 (having a characteristic yielding strength of 500 MPa).



Figure 3: Seven storey reinforced concrete building constructed in Cardington.

Table 1: Configurations of flexural reinforcement in the in-situ building at Cardington by Goodchild (2000)

Floor N°	Flexural reinforcement	Design approach	Tonnes /floor
1	Traditional loose bar	Elastic Design	16.9
2	Traditional loose bar	Elastic Design	17.1
3	Rationalised loose bar	Elastic Design	15.3
4	Blanket cover loose bar	½ Yield Line design ½ Elastic Design	14.5 23.2
5	One-way mats	Elastic design	19.9
6	Blanket cover 2-way mats	FE Design	25.5

During the BRE study, many different design and detailing methods were used and compared. From the study reported by Goodchild (2000), the yield line method resulted in the most economic reinforcement (Table 1). This method is based on the use of uniformly distributed bars, one or two way spanning mats, which are very attractive as flexural reinforcement. Top rebars should be concentrated over supports to control cracking. For this reason, the SFRC solution is also based on the yield line method.

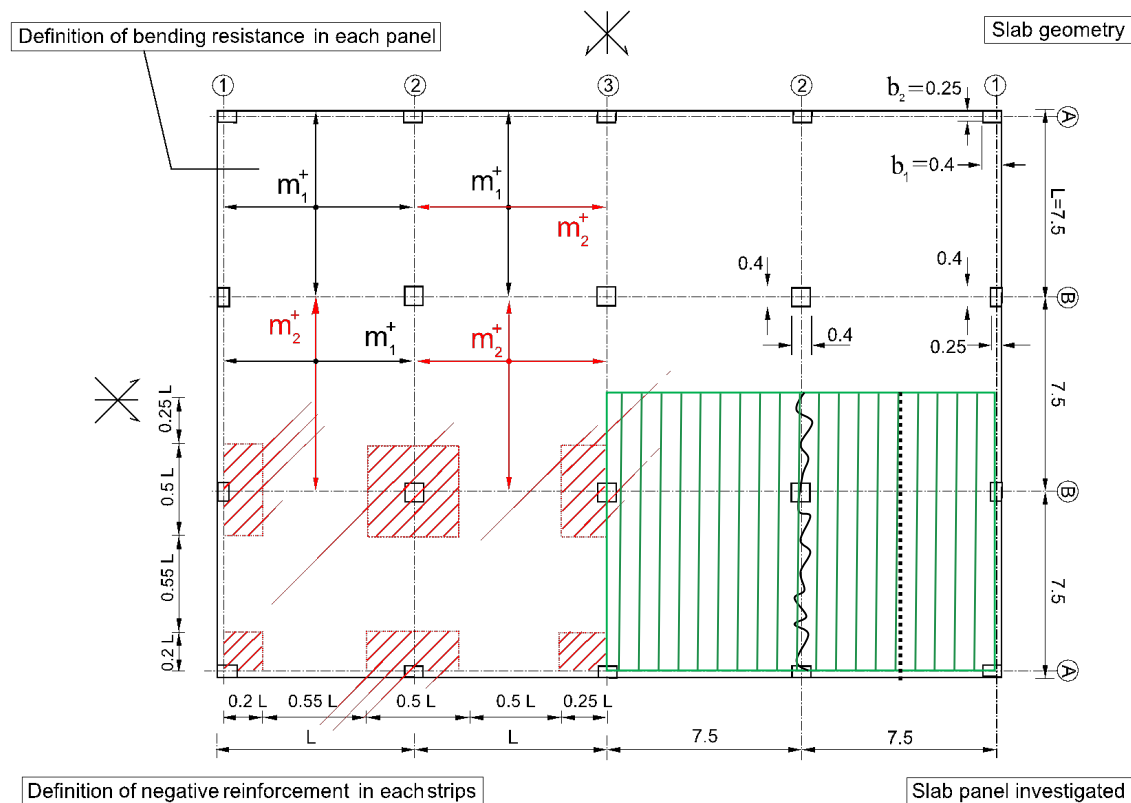


Figure 4: Plan view of the slab, reinforcement distribution and grid reference lines. Reinforcement for negative moment is located in red, diagonally cross-hatched zones. In the green, vertically cross-hatched zone, the symmetric failure mechanism investigated is shown (crimped line = negative moment; dotted line = positive moment).

Table 2: Design procedure

Design variables

- t : thickness of the slab;
- FRC classification: 3c;
- ρ_s : positive reinforcement ratio;
- ρ_s' : negative reinforcement ratio;
- ρ_p : punching reinforcement ratio.

Equations governing structural design

- $L/t(K, \rho_s, \rho_s', f_{ck})$; Eqs. 7.6-22a and 7.6.22b; deformability check;
- $M_{rd}(t, f_{Ftuk}, \rho_s, \rho_s')$; bending strength of the section;
- $w_{cr}[M(\chi)]$; crack opening deduced from non-linear plane section analysis;
- $V_{rd}(t, f_{Ftuk}, f_{ck}, M_{rd}, \rho_p)$; Eqs. 7.7-17;
- Robustness of the flat slab.

Starting the design with the deformability check, a simplified approach using a span/depth ratio is chosen. Therefore, starting from the grid line layout that defines the span length, the minimum slab thickness necessary to withstand the deformability requirements is determined. The span/depth ratio is a function of the reinforcement ratio required at mid-span to resist the moment due to design loads and it does not take into account FRC contribution. Having chosen to adopt a yield line method, it is known from Park and Gamble (1980) that beamless (flat slab)

floors subjected to a uniform load have two types of yield line pattern. One involves general folding of the floor and the other involves local slab collapse around the column. In this phase of the design, in order to satisfy Equations 7.6-22 a-b, the folding of the floor is assumed. In this way, it is possible to find a minimum thickness and a reinforcing ratio able to satisfy deformability and ULS. This phase is the same for both SFRC and RC solutions: in fact, the same thickness is used. After that, it is possible to evaluate the contribution of FRC for the bending resistance, in order to re-define the necessary reinforcement ratio. As this procedure does not modify the thickness, the deformability check does not change. This is true if a crack pattern similar to the one of the RC solution is guaranteed. It should be noted that, since this method considers only possible collapse mechanisms, the serviceability limit state (i.e. crack opening) has to be checked.

Once the thickness is assumed, the rebar layout of the Cardington slab is calculated considering two main folding collapse mechanisms. The difference between the two mechanisms is related to the boundary conditions (Figure 5). The first one is a perimeter condition in which there is a free edge and the other one is a central condition in which the whole perimeter is clamped. This difference modifies the yield line solution and, therefore, the required bending moment resistance for given design loads. The slab could be divided into internal panels, corner and edge ones. The rebar layout is defined for these typical conditions.

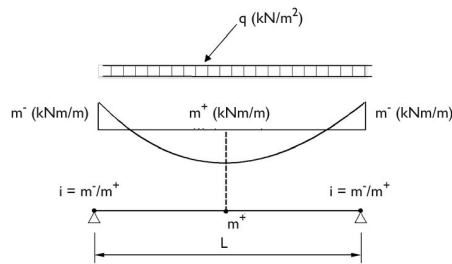
In the design phase, the ratio between the average ultimate negative to positive moments is adopted and it is set equal to $i = m_j^- / m_j^+ = 1.85$ with $j = 1, 2$. This value is simply chosen to follow the initial distribution of bending moments related to an elastic behaviour of the slab. Similarly, from elastic theory it is known that the negative moments in the slab near the columns are very high and yielding of steel may occur there under the service load. To ensure the correct behaviour at SLS, the reinforcement is placed according the elastic bending moment. Thus, the positive moment steel is distributed uniformly (or nearly uniformly) across the panel, whereas the rebars for negative moment are concentrated entirely or primarily in the column strips. By using this approach, the whole slab is designed by carrying out the check for local failure, crack opening for each panel, punching and robustness evaluation. In the following, the design procedure for the corner panel (that is the most severely stressed) is reported. From folding type failure (Figure 5), it is possible to define the yield line moment considering:

$$m_1^+ = \frac{q_{sd} \cdot (L - (0.5 \cdot b_1 + 0.5 \cdot b_2))^2}{2 \cdot (1 + (1 + i)^{0.5})^2} = 52.4 \cdot kN \frac{m}{m} \quad (18)$$

The contribution of FRC is calculated by considering the assumed performance class (3c), a K_{Rd} value of 1.2 and a simplified post-cracking constitutive law (Eq. 5.6-4 of MC 2010):

$$m_{Rd}^{FRC} = K_{Rd} \cdot \frac{f_{R3k}}{\gamma_{FRC}} \cdot \frac{t^2}{6} = 22.5 \cdot kN \frac{m}{m} \quad (19)$$

Therefore, it is possible to redefine the necessary amount of reinforcement. This results in rebars Ø 10 spaced 20 cm at the bottom, in two directions. Regarding the top reinforcement, it is better to concentrate it around the column. In this example, it was decided to concentrate it over a width of 0.5 times the span L (Figure 4); in this way the distribution of the rebars is easy, as only a bottom layer is in all the sections. With this assumption, rebars Ø 18 spaced



Case	Diagram	Formula
1		$m_1^- = \frac{qL^2}{2(1 + \sqrt{1+i})^2}$
2		$m_2^- = \frac{qL^2}{8(1+i)}$

Figure 5: Equations for one-way slabs to determine the midspan yield line moment (m ; Park and Gamble, 1980).

12 cm are needed. To show how FRC contributes in all sections, also if no rebars are spread in the whole span, the calculation for the middle line of the green field is reported (Fig. 4, vertical line (2) connecting columns 2A and 2B). Taking into account only half of the slab for symmetry, the integral of the negative moment (due to the load along these yield lines - crimped line in Figure 4) can be written as:

$$M^- = m_1^- \cdot \left(\frac{b_2}{2} + 1.5 \cdot L \right) = i \cdot m_1^+ \cdot \left(\frac{b_2}{2} + 1.5 \cdot L \right) = 1102.7 \cdot \text{kNm} \quad (20)$$

By considering the contribution of SFRC along all these lines, it can be obtained:

$$\Delta M^- = M^- - m_{Rd}^{FRC} \cdot \left(\frac{b_2}{2} + 1.5 \cdot L \right) = 846.7 \cdot \text{kNm} \quad (21)$$

Therefore, the contribution that is needed from rebars is:

$$m_1^{RC} = \frac{\Delta M^-}{\frac{b_2}{2} + 0.7 \cdot L} = 157.5 \cdot \text{kN} \frac{\text{m}}{\text{m}} \quad (22)$$

Also a local failure mode is checked for each panel, in particular those shown in Figure 6.

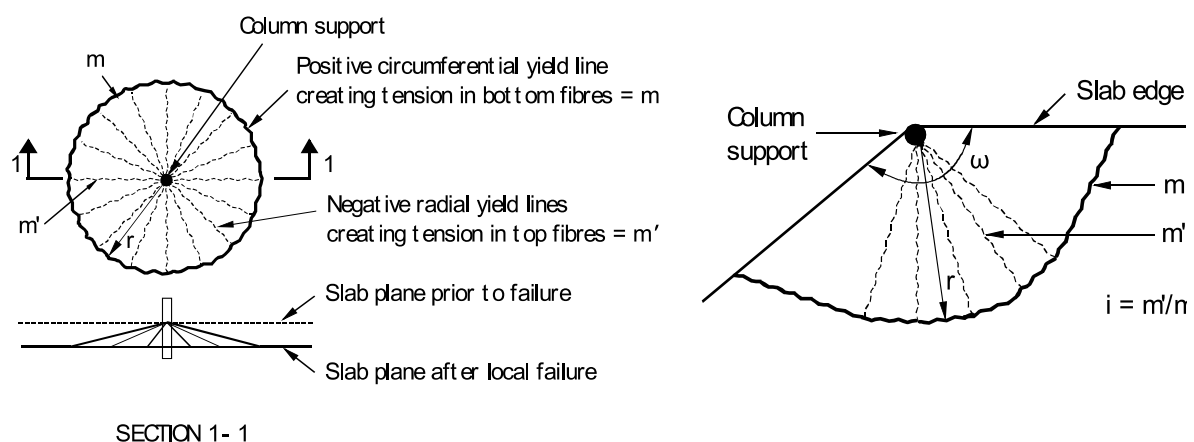


Figure 6: Local failure pattern (Park and Gamble, 1980).

The serviceability check is done by assuming an exposure class equal to XC1. Therefore, according to Chapter 7.6 of MC 2010, a maximum crack opening equal to 0.3 mm (referring to a quasi-permanent load) must be guaranteed. This quasi-permanent load corresponds to a uniformly distributed load for Category C (Table 4.5-1 of MC 2010; $7+0.6 \cdot 3.5=9 \text{ kN/m}^2$). The bending moment distribution in the slab corresponding to this load level is calculated using a FEM elastic analysis. The evaluation of the crack width is done by adopting Equations 7.7-22, 7.7-23 and 7.7-23 of MC 2010. A crucial point for the correct estimation of the crack opening is the definition of the tension in the rebar at the cracked section, considering the beneficial effect of FRC. A non-linear plane section analysis for FRC solution is performed. With the aim of considering only the pull out effect of fibres, neglecting the post peak matrix contribution during initial crack propagation phase (as for the RC solution), a linear model is used in the analysis by using the recommendations proposed by di Prisco et al. (2013). The crack width calculated for the FRC slab results in a reduced value of 0.1 mm which is three times lower than the maximum allowed by MC 2010. This means that the FRC solution should be able to increase the durability performance of the structure. A summary of the calculated results is reported in Table 2, while in Figure 8 is shown, as an example of the sectional behaviour above column B2, a comparison between the FRC and RC solution in terms of crack opening. Since the RC design was originally done with $i=1$, the bending resistance is different with respect to the FRC solution. Therefore, it is more correct to refer to a fictitious solution in RC; in this case, the crack width is 25% higher than for FRC.

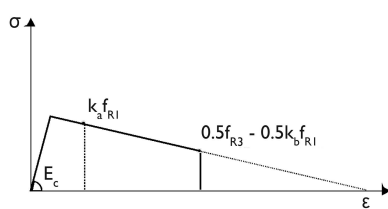


Figure 7: Elasto-Linear model for FRC as defined in di Prisco et al. (2013).

Figure 8: RC vs. FRC solution above column B2 in terms of crack opening.

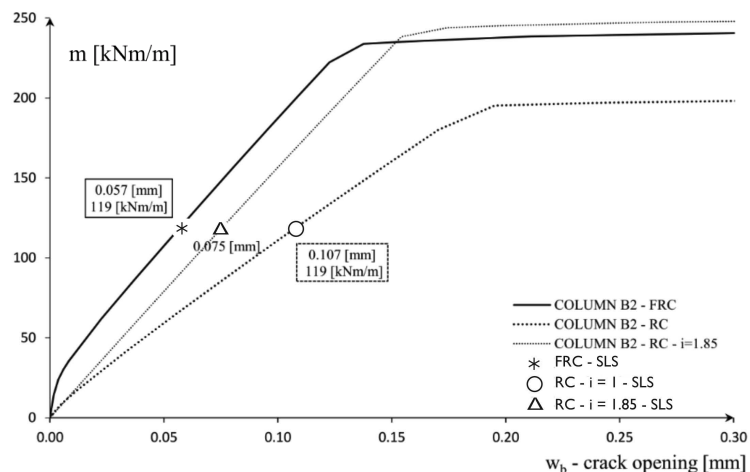


Table 2: FRC solution: bending moments and crack opening calculated for quasi-permanent loading combination ($q_{sk}=9 \text{ kN/m}^2$)

	POSITION					
	COLUMN B1*	COLUMN B2*	COLUMN B3*	CORNER	EDGE	CENTRAL
$m_{rd} [\text{kN m/m}]$	-83	-200	-107	55.5	36.6	36.6
$m_{rk} [\text{kN m/m}]$	-94	-242	-132	70	48	48
$m_{sk} [\text{kN m/m}]$	-49.8	-118.9	-93	45	31	14
$w_k [\text{mm}]$	0.05	0.06	0.1	0.1	0.05	0.01
$ m_{rk,j}/m_{rk,2}^+ $	1.95	5	2.75	1.45	1	1

(*) refer to Figure 4.

As it is well known, a critical condition for flat slabs is the punching bearing capacity. The presence of FRC results in a performance increase and a reduction of the required reinforcement. Here the design equations for the B2 column, which is the most severely loaded one, are reported. As mentioned in Chapter 7.3.5 of MC 2010, there are different levels of analysis when using the critical shear crack theory (CPCT), as proposed by Maya et al. (2012). The level I approximation is typical for a preliminary design while, for a refined design, a level II approximation is more suitable. The first level is used with the aim of checking if the thickness of the slab is reasonable with respect to the punching shear strength and if punching shear reinforcement is needed. From this analysis it can be highlighted that the slab needs punching reinforcement; moreover, it is also clear that the presence of FRC provides a significant contribution, to be estimated as 30% of the acting punching load. To design the punching reinforcement, a level II approximation is adopted. The main difference between these two levels of approximation is the way to evaluate the rotation of the slab at the connection with the column. In fact, in case of significant bending moment redistribution, it is possible to take into account this aspect with the introduction of a ratio m_{Ed}/m_{Rd} (Eq. 7.3-75 of MC 2010). Notice that m_{Ed} is the acting average moment (per unit length) for calculation of the flexural reinforcement in the support strip while m_{Rd} is the average resistant flexural bending moment (per unit length) in the support strip. Due to the criteria adopted during the design of the support strip, the moment resistance is equal to 2 times m_{Ed} . This ratio is lower in reality since, as shown in Equation 21, there is also a FRC contribution in the middle strip. Therefore, this ratio ranges between 1 and 2, according to the ratio between the m_{FRC} contribution and the m_{Ed} . In particular, this ratio is given as:

$$\frac{m_{Ed}}{m_{Rd}} = \frac{m_1}{m_{Rd}} = \frac{\alpha - 1}{(\alpha \cdot k - 1)} \quad (23)$$

where:

$$k = \frac{m_{Rd}^{FRC}}{m_1} = \frac{22.5 \cdot kNm/m}{1.85 \cdot 52.4 kNm/m} = 0.23, \text{ and } \alpha \text{ is defined as:}$$

$$\alpha = \left(1 - \frac{L_{Rd}}{L_{tot}}\right) = \left(1 - \frac{(0.7 \cdot L + \frac{b_2}{2})}{(1.5 \cdot L + \frac{b_2}{2})}\right) = 0.53$$

In particular, the ratio of Equation 23 is equal to 0.53 and brings to an increase of punching resistance of 57% following level II approximation and considering the value of the ratio computed in Equation 23. In any case, the punching resistance is not sufficient and a 8 mm @160x160 rebar (or studs) is required (Figure 9). The RC solution, as designed in the BRE project, is summarized in Table 3 and shown in Figure 10, together with the FRC solution. By considering the RC and the FRC solutions, the steel content results in about 92 kg/m³ and 77 kg/m³ (including the fibre content), respectively. Therefore, the FRC solution leads to a reduction of about 16% of bending reinforcement, in addition to the reduction of punching reinforcement. Moreover, FRC allows a significant reduction of the labour time for placing the reinforcement. To evaluate the robustness of the FRC solution, different conditions in which one or more columns are eliminated, are studied. The failure mechanisms investigated are shown in Figure 11, based on the Johansen approach (1967). They are compared for a uniformly distributed load equal to the quasi-permanent load condition, adopting the coefficients for Category C in Table 4.5-1 of MC 2010 (9 kN/m²).

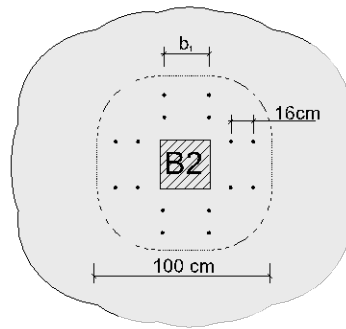


Figure 9: Layout of punching reinforcement needed on column B2.

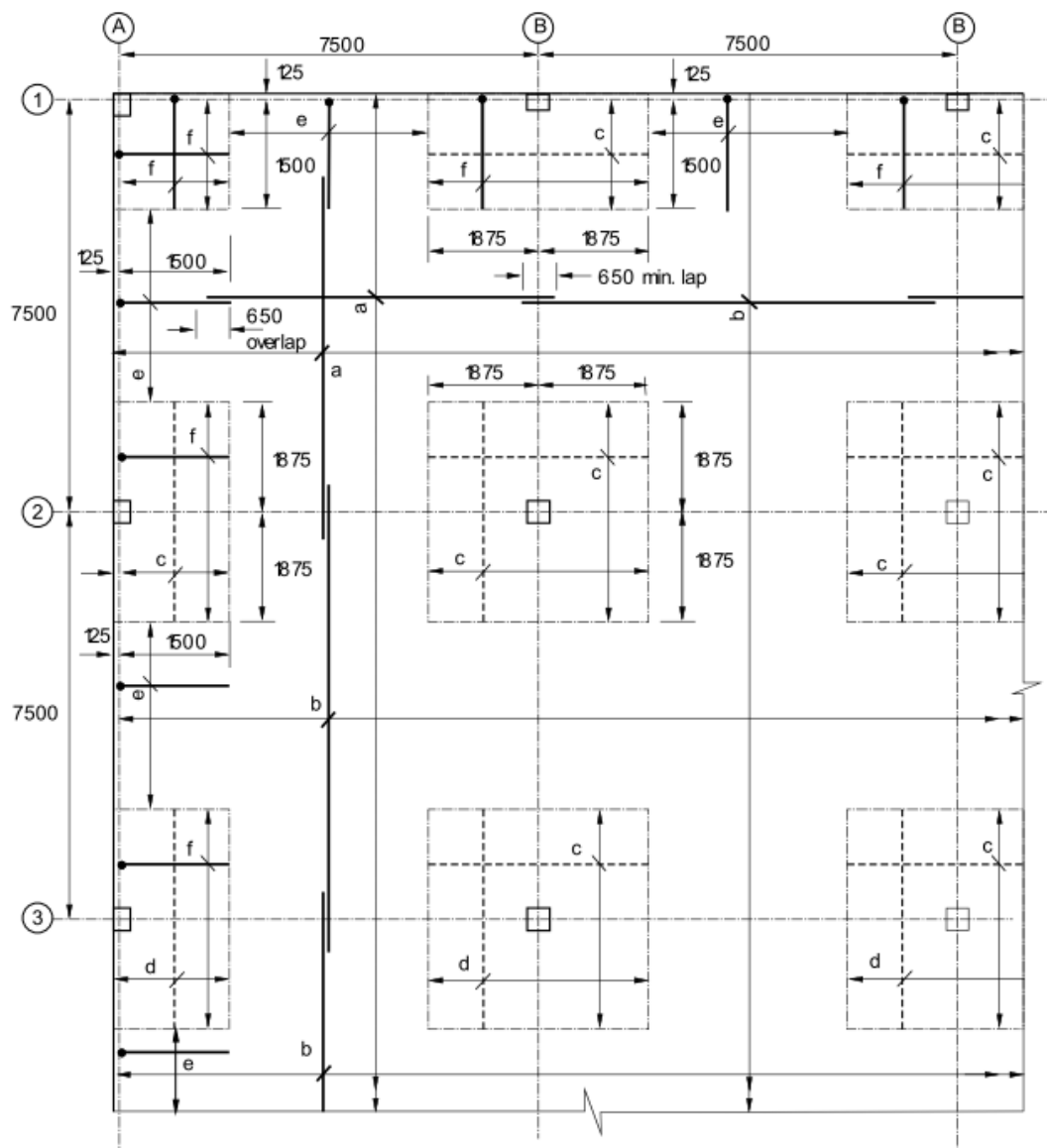


Figure 10: Reinforcement layout of flat slab reference.

Table 3: Rebars adopted for traditional and FRC solutions

bar marks	RC SOLUTION			FRC SOLUTION		
	Diameter	spacing	shape	diameter	spacing	shape
a	16	175	Straight	10	200	Straight
b	16	300	Straight	10	450	Straight
c	16	100	Straight	18	120	Straight
d	16	150	Straight	16	200	Straight
e	12	250	U bar	12	400	U bar
f	12	125	U bar	12	150	U bar

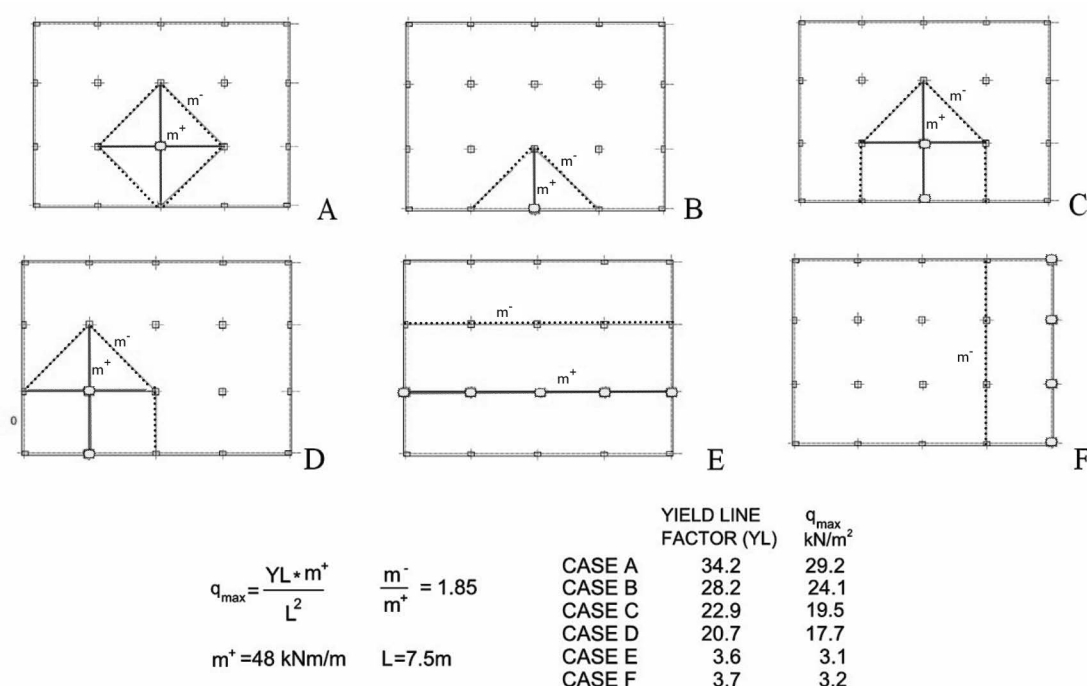


Figure 11: Investigated failure mechanisms due to the loss of one or more columns.

In order to simplify the computational work, a reinforcement equal to m_2 is assumed in all the panels (Figure 4), thus considering a $m_{rk}=48 \text{ kNm/m}$ and a ratio between positive and negative moment equal to 1.85. This hypothesis is on the safe side as, in reality, the bending resistance of the section is everywhere equal or higher. Due to the high degree of redundancy of the structure, the robustness is very high and is guaranteed by the presence of FRC which is able to react in the same way if subjected to positive or negative bending moment. In this way, the yield line analysis of different failure modes can take into account the beneficial effects of diagonal negative plastic hinges. This situation is completely different in case of the RC solution, because it was decided to adopt uniformly distributed rebars only in the bottom layer. The RC slab is able to react only above the column head and not along a straight line that does not cross any column (Figure 11). It should be noticed that only cases E and F bring to collapse, even if in case E the chain effect is able to save the structure and in case F the contribution of the upper columns along the edge is not taken into account.

5. Concluding remarks

According to the new Model Code 2010, FRC represents a new opportunity to simplify and make more efficient the reinforcement of concrete structures. In order to show the potentialities of this material, two basic problems are considered in this paper: a wide-shallow beams and a suspended slab. In both cases, a cost reduction of about 16-18% of steel reinforcement is demonstrated for FRC elements, in addition to a reduction of labour time for placing the reinforcement. Moreover, FRC enhances 1) structural durability due to the reduction of the crack opening; 2) fire resistance due to the good performance of steel fibres at high temperatures and 3) the robustness in statically indeterminate structures. In the case of the suspended slabs, the beneficial effect of the optimized reinforcement is well evidenced and only the presence of top rebars all over the slab could offer a similar robustness in a usual RC structure. However, this means a significant increase of rebars at the extrados with a cost increment of about 25% for the RC solution. A general conclusion about this comparison can argue that, due to the isotropic randomly distributed fibre reinforcement, fibres can act in any direction in the plane, giving a significant contribution both as positive or negative bending reinforcement in any point (that significantly enhance robustness). Moreover, it can contribute also to punching and shear at the same time. This means that fibres are really able to contribute along any cracked plane in a three dimensional reference and, in statically indeterminate structures, they appear very efficient in terms of resistance and robustness when compared to bars which provide only a significant contribution in one direction.

6. Acknowledgements

A special acknowledgement goes to MS. Eng. Andrea Bassani and Dr. Antonio Conforti for their precious contribution concerning the computations related to the cases study analysed.

7 References

- ACI Committee 544 (1996), *Design considerations for steel Fiber Reinforced Concrete*, ACI 544.4R-88, American Concrete Institute, ACI Farmington Hills.
- Barros J. (ed.), *Fibre Reinforced Concrete: challenges and opportunities*, *Proc. of the 8th Int. Symp. Bagnaux*, France, RILEM Publications S.A.R.L., PRO88., 2012.
- Conforti, A., Minelli, F., and Plizzari G.A. (2013), "Wide-shallow beams with and without steel fibres: a peculiar behaviour in shear and flexure", *Composites Part B: Engineering*, V. 51, pp. 282-290, <http://dx.doi.org/10.1016/j.compositesb.2013.03.033>.
- Cuenca, E., and Serna, P. (2013), "Shear behaviour of prestressed precast beams made of self-compacting Fiber Reinforced Concrete", *Construction and Building Materials*, V. 45, pp. 145-156, <http://dx.doi.org/10.1016/j.conbuildmat.2013.03.096>.
- Cuenca, E., and Serna, P. (2013), "Failure modes and shear design of prestressed hollow core slabs made of Fiber-Reinforced Concrete," *Composites Part B: Engineering*, Vol. 45, No. 1, ISSN 1359-8368, pp. 952-964.
- di Prisco M., Felicetti R., Plizzari G. (eds) (2004), *Fibre Reinforced Concrete, BEFIB 2004*, Bagnaux, France, RILEM Publications S.A.R.L., PRO39.

- di Prisco, M., Colombo, M., Dozio, D. (2013), Fibre-reinforced concrete in *fib* Model Code 2010: principles, models and test validation, *Structural Concrete* 14 (4), 342-361.
- di Prisco, M., Martinelli, P. (2014), A numerical approach for the evaluation of the structural redistribution coefficient K_{Rd} , *Comp. Modelling of Concrete Structures – Proc. EURO-C 2014*, Vol. 1, 2014, pp. 503-512, St. Anton am Arlberg; Austria.
- di Prisco, G., Plizzari, G., Vandewalle, L. (2009), Fibre reinforced concrete: new design perspectives, *Materials & Structures*, 42(9), 1261-1281.
- Dinh, H.H., Parra-Montesinos, G.J., and Wight, J. (2010), “Shear Behaviour of Steel Fibre-Reinforced Concrete Beams without Stirrup Reinforcement”, *ACI Structural Journal*, V. 107, No. 5, pp. 597-606.
- Dozio, D. (2008), *SFRC structures: identification of the uniaxial tension characteristic constitutive law*, PhD thesis, Department of Structural Engineering, Politecnico di Milano, Sarrylink Ed., ISBN 978-88-96225-14-1.
- fib* Bulletin 65 (2012), “*Model Code 2010 - Final draft*”, Volume 1, 350 pages, ISBN 978-2-88394-105-2.
- fib* Bulletin 66 (2012), “*Model Code 2010 - Final draft*”, Volume 2, 370 pages, ISBN 978-2-88394-106-9.
- Gettu R. (ed.) (2008), Fibre Reinforced Concrete: design and applications, *BEFIB 2008*, Bagneux, France, RILEM Publications S.A.R.L., PRO60.
- Goodchild, C.H. (2000), *Rationalisation of flat slab reinforcement*: Crowthorne, British Cement Association, Ref.97.376. 210 pp.
- Park, R. & Gamble, W. L. (1980), *Reinforced concrete slabs*. New York, John Wiley & Sons, 618 pp.
- Hedebratt, J., Silfwerbrand, J. (2013), *Full-scale test of a pile supported steel fibre concrete slab*.
- Maya, L.F., Fernández Ruiz, M., Muttoni, A., Foster, S. (2012), Punching shear strength of steel fibre reinforced concrete slabs, *Engineering Structures*, Vol.40, pp. 83-94.
- Minelli, F., Conforti, A., Cuenca, E., and Plizzari, G.A. (2014), “Are steel fibres able to mitigate or eliminate size effect in shear?”, *Materials and Structures*, V. 47, No. 3, pp. 459-473, doi: 10.1617/s11527-013-0072-y.
- Johansen, K.W. (1967). *Yield Line Theory*, London: Cement and Concrete Association.
- Reinhardt H.W., Naaman A.E. (eds) (2007), *High Performance Fibre Reinforced Cement Composites (HPFRCC5)*, Rilem Publication S.A.R.L., PRO53.
- Rossi P., Chanvillard G. (eds) (2000), 5th RILEM symposium on Fibre Reinforced Concretes, *BEFIB 2000*, RILEM Publications S.A.R.L., Cachan, France.
- Vasanelli, E., Micelli, F., Aiello, M.A., Plizzari, G. (2013), Long term behaviour of FRC flexural beams under sustained load, *Engineering structures*, Vol. 56, pp. 1858, 1867.

Design-based approaches for fibre-reinforced concrete: An overview of ACI committee 544 activities

Barzin Mobasher

Arizona State University, Tempe, USA

Abstract

The ACI Committee 544 on fibre-reinforced concrete (FRC) has been involved in development and dissemination of technical information for nearly a half century. A key advantage in using FRC is the reduction in construction time compared to the traditional reinforcing bars or welded wire mesh. Application areas for FRC have extended to areas where high early strength and ductility are important and include pavement, shotcrete and structures such as bridge deck slabs, or rock slide stabilization. In these cases, the material properties must be measured using experimental test data obtained from an experimental program. Test results must be analysed in order to obtain effective stress strain responses that can be incorporated in analytical, or computer simulation. A list of examples including wall panels, hydraulic structures, airport pavements, and industrial floor overlays are described. To maintain integrity without collapse, such structural elements need to be designed with proper material models and analysis tools discussed.

Keywords

Ductility, fibres, fibre-reinforced concrete, flexural tests, tension tests, stress-strain, toughness, back-calculation.

1 Introduction

The ACI Committee 544 on fibre-reinforced concrete has been involved in development and dissemination of technical information for nearly a half century. Formed in 1965, the committee enjoys a membership consisting of academicians, engineers & designers, construction industry practitioners, and fibre manufactures. The membership's continuity of experience in addition to a steady supply of new research results, recent reports and educational tools has been instrumental in its success.

The primary directions of Committee 544 are defined in terms of three major interacting areas which include: a) education, codes and specifications, applications, sustainability and life cycle cost, b) fundamentals of testing, modelling, mechanical and physical properties, and c) durability and serviceability aspects. The committee's composition was rearranged about 12 years ago into the current structure of six subcommittees that operate within the main ACI 544. These include: A) FRC production and applications, B) FRC education, C) Testing, D) Structural uses, E) Mechanical properties, and F) Durability. Some of the recent activities of the ACI 544 are presented in the following sections.

2 Thrust areas

2.1 Sustainability and life cycle cost of FRC materials

A key advantage in using FRC is the reduction in construction time compared to the traditional single or double layers of conventional reinforcing bars, welded wire mesh, or stirrups for shear reinforcement. The reduction of traditional welded wire reinforcement or layers of reinforcing bar improves mobility and personnel safety. By switching to FRC components, time savings of several weeks for large scale projects can be obtained. Since the fibre-reinforced concrete can be directly pumped to the elevated slabs, canals, or bridge decks, the use of heavy construction equipment for lifting and placement of reinforcing bars is eliminated. FRC systems improve the architectural freedom with non-orthogonal shapes and floor plans, which may otherwise be difficult with reinforcing bar layouts and forming. Savings in crane costs and scheduling aspects is obvious because the reinforcement is pumped and with self-consolidating mixtures, installation avoids vibrations, eliminates poker holes, and reduces noise pollution.

Jobsite simplifications improve the safety as reduction of labour-intensive tasks reduce the number of personnel involved. Moreover, by reducing the requirements for concrete cover, additional reductions in material and labour are obtained due to thinner sections as the capacity is verified by moment, shear, and deflection calculations. To support these calculations work is conducted to document potential benefits of FRC materials in terms of their sustainability and reduction of life cycle cost. Advantages of using the FRC in sustainable designs are addressed in terms of speed of construction, strength, ductility, shrinkage crack width control, and long term durability (Mobasher, 2011; di Prisco et al. 2004).

2.2 Development of simplified constitutive models for design applications

Computational models provide opportunities for robust analysis and design with FRC materials. In order to develop standardized procedures for design with FRC materials, test methods must be calibrated to obtain material properties. Extensive work in the development, design, analysis, and fieldwork with FRC has been done and efforts are under way to develop design guides by various research groups (di Prisco et al. 2000, 2003; Rilem, 2000; Vandewalle, 2003; Soranakom and Mobasher, 2008). Formulations for simplified design and analysis that incorporate stress-strain relationship based on material behaviour are proposed. The ACI Committee 544 is working on the implementation of material models in finite element and elastic plastic solution methodologies in order to close the gap between properties, analysis, modelling, and design. Closed form relationships based on plastic design of reinforced concrete are useful in calculation of the load deflection response of a nonlinear SHCC and SSCC composites (Soranakom and Mobasher, 2007, 2008).

2.3 Durability- and serviceability-based design

It is probable that given the FRC technology available today, a majority of the design cases we face can be improved in their durability, ductility, and serviceability. One of the main advantages of the FRC is the reduction in the number of joints and use of continuity as a means of moment redistribution and redundancy of the structure. The shrinkage control enhancement produced by incorporation of fibres promotes the application of joint-free slabs and environmental structures and bridge decks (Silfwerbrand, 2004; Grzybowski, Shah, 1990). Many of the long term performance issues dealing with the maintenance of the joints are eliminated by reducing their overall number with typical construction joint spacing specified at up to 45 meters, as much as three times the conventional methods. Increasing the joint spacing affects parameters such as curling, joint movement, and corner cracks which are dominant failure modes. In addition to these benefits, significant shrinkage cracking control due to fibre addition directly results in a reduced number of construction joints (Mangat, Azari, 1988). Additional savings are also obtained in the preparation of the drawings as fibres that replace reinforcing bars do not need to be detailed (Destrée, 2008, a, b). Many of the efforts of ACI 544 are currently geared towards development of technical notes to address the various durability and serviceability objectives.

3 Emerging technology reports

3.1 Material post-peak property characterization

Design procedures for FRC require simplified equations to account for the contribution of fibres to the tensile response, cracking strength, and post crack softening response. FRC mixtures exhibit tensile strain softening and fibre contribution to the bridging force is expressed in terms of an average smeared pullout-slip response. The bridging force, represented as an average effective tensile stress-crack width relationship is used as a material property using nonlinear spring elements to simulate the residual capacity of a cracked section. (Naaman and Reinhardt, 2005; Barros, 2005).

The purpose of the ACI ET-Report titled: “Report on Indirect Method to Obtain a Stress-Strain diagram for Strain Softening Fiber-Reinforced Concretes (FRCs)” (2014) is to present curve fitting and back-calculation approaches that use flexural data to compute tensile response. The main approach for flexural testing was developed by RILEM, (TC 162-TDF 1995) and later modified through EN-14651, however the tensile properties are obtained using finite element back-calculations and results are obtained by curve fitting the data from sample size studied (Vandewalle and Dupont, 2003). These approaches use empirical and inverse-analysis methods and show that the back-calculated post peak residual tensile strength is about 30%-37% of the elastically equivalent flexural residual strength for specimens with different fibre types and volume fractions. The report introduces the parameters that define the stress-strain diagram (Soranakom, Mobasher, 2009). Calculation procedures for obtaining tensile and residual flexural strengths from beam test data are presented using closed form solutions for the moment-curvature response that are derived explicitly. Load vs. deflection in addition to expressions for calculating neutral axis depth ratio, k , moment, and curvature are obtained in closed form. The stress crack width relationship is expressed in terms of parameterized

coefficients (Soranakom, Mobasher, 2008). Coefficients proposed by RILEM Committee TC 162-TDF are validated with other flexural tests as well as ASTM C1609/C1609M-10 which do not report coefficient values applicable for design. The ACI report proposes an approach to obtain effective tensile data and compare the results with different flexural tests (ASTM C1609-10, 2010). Statistical aspects of back-calculated test data are addressed under separate publications (Mobasher et al., 2014).

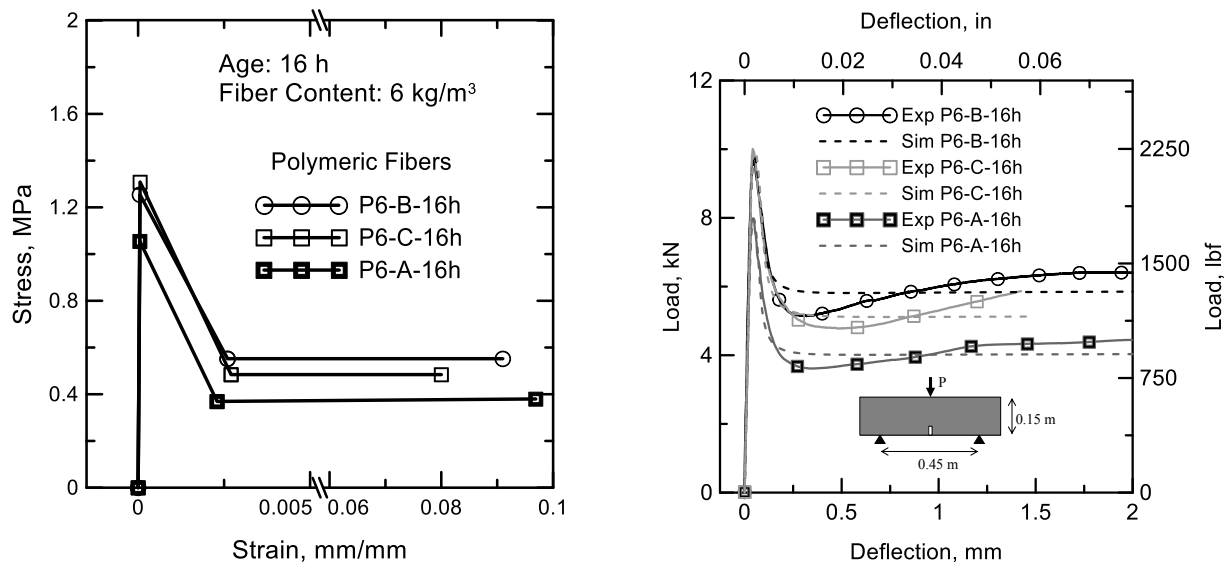


Figure 1: Effect of fibre type on: (a) back-calculated tensile stress strain response and (b) experimental and simulated load deflection response for polymeric fibres.

Figure 1 shows the back-calculation technique to obtain the tensile properties of polymeric fibres (Bakhshi, et. al, 2014). The specimens were tested by flexural test methods ASTM C1609 which measures the post cracking characteristics in terms of Average Residual Strength (f_{150} values) (ASTM 1609). It is imperative to note that the f_{150} stress is not an equivalent elastic stress and cannot be associated with the post-crack tensile strength. Correlation between the residual strength and f_{150} from ASTM test methods are shown in figure 2 and overestimate the residual strength by the ARS values by a factor of 2.95 which correlates very well with the reduction factor of 0.37 used by Rilem (Soranakom and Mobasher, 2010).

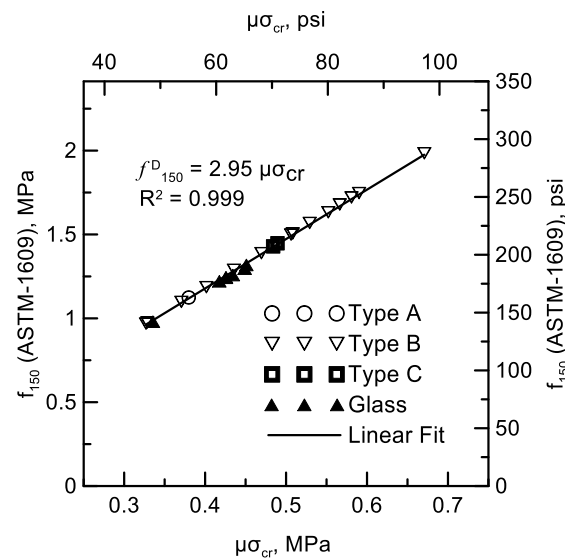


Figure 2: Comparison of residual strength ($\mu\sigma_{cr}$) with ASTM-1609 residual parameter.

3.2 Design of FRC in elevated slabs and panels

Use of fibre-reinforced concrete in the structural design of pile-supported slabs and floor decks is quite common for areas where soil-structure interaction may create differential settlement, cracking, or long-term serviceability problems. It is estimated that 20% of industrial concrete floors are supported on piles (Reddy, 2003). The design procedure for SFRC by TR 34 was developed in 1992 in UK (Concrete Society TR 34, 2003) and has been used on several million square meters (Dhir, Henderson, 1999; Chandler and Neal, 1988). The SFRC slabs resist high moment intensities as well as high shear and punching shear stresses. The construction of slabs on closely spaced piles is referred to as elevated ground slabs which depending on the loading intensity and pile capacity, span-to-depth ratios between 8 and 30 are used. These slabs can be subjected to moderately high loading such as concentrated point loading of up to 44 kips (150 kN), uniformly distributed loadings of 1000 lb/ft² (50 kN/m²), and dynamic loadings.

The ACI ET document addresses the methodology for analysis, design, and construction of steel fibre-reinforced concrete slabs supported on piles, or columns (E-SFRC). A design approach is presented for cast in place above columns or beams and extended to the elevated slabs and multi-story building applications. The basis for the materials use is a relatively high dosage of steel fibres (85-170 lb/yd³ (50-100 kg/m³)) recommended as the primary method of reinforcement. Standard procedures for obtaining material properties and development of finite element models for structural analysis of the slabs are discussed and methods of construction, curing, and full-scale testing of slabs are also presented. A typical example is shown in Figure 3 for a multi-storey high rise condominium building showing sections that are generally flat with a span to depth ratio of up to 30. A high dosage of steel fibres (50-100 kg/m³) is used as the sole method of reinforcement. Large bay areas are possible due to shrinkage control which directly reduces the construction joints since shrinkage crack control is alleviated by the fibre reinforcement (Destrée, 2008; Mobasher, Destrée, 2011).



Figure 3: Applications of elevated flat slab using steel fibres as primary reinforcement.

3.3 Precast FRC tunnel sections

One of the most successful applications for FRC materials is in the precast concrete segments that are predominantly used as the support method in tunnel boring machine (TBM) bored tunnels in soft ground and weak fractured rock. In one-pass lining system where the segments provide the initial and final ground support, the segments must be designed to withstand loading from the ground and groundwater as well as the loads imposed during production, transportation, and construction. Tunnel segments are generally reinforced with conventional rebars to resist the tensile stresses both at Serviceability (SLS) and Ultimate Limit States (ULS) (Bakhshi Nasri, 2014), however their use results in large crack widths during demolding, storage, transportation, handling, and installation phases. Other issues include but are not limited to high labour cost and time associated with the assembly and placement of rebar cages during the precast segment manufacturing process, and durability issues when segments are exposed to corrosive and severe exposure conditions. FRC has been used in numerous projects for the construction of tunnel precast segmental lining (De la Fuente et. al. 2012, Hansel, Guirguis, 2011). Segmental linings for smaller diameter tunnels have been reinforced only with steel fibres at a dosage rate between 40 and 100 lb/yd³ (25 and 60 kg/m³). With the introduction of high performance FRC, fibres (min. 67 lb/yd³ or 40 kg/m³) are used as the sole reinforcement. In other situations, fibre and steel bars are used as reinforcement system.



Figure 4: Precast Concrete Segments prior to transportation to Tunnel Boring Machine.

4 Technotes to address applications development

Finite element approach is used for analysis and design of a given structure or inverse analysis to obtain a prescribed load-deflection serviceability criteria and/or structural ductility characteristics. Several types of panel tests (EFNARC, 1996; Bernard, 2000; ASTM C1550-02, 2002) have been used to simulate the stress state in plate type specimen and evaluate the flexural capacity of FRC. Test results produce a load deflection response which can be used to calculate the energy absorption as a material index. As the size, thickness, and fixity of the panel changes, the load, deflection, and energy absorption also change.

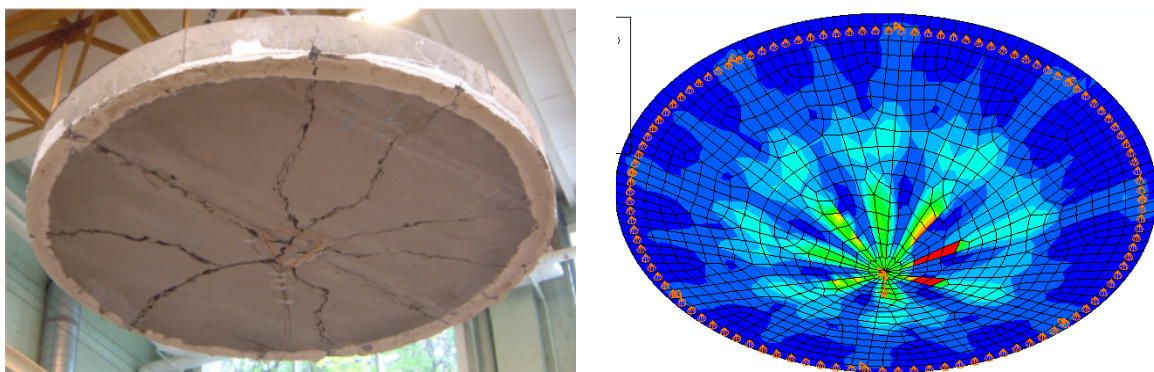


Figure 5: Large scale ($D=2$ m) round panel tests for back-calculation of materials properties (Mobasher, Destrée, 2011).

Figure 5 shows the finite element simulations developed for a panel test (ASTM C1550, 2002; Bernard, 2000). Test conducted under different boundary conditions help measure the flexural capacity of FRC in 2-D. Using elastic-plastic failure modes simulation of the stress state, localization, and deformation patterns in an indeterminate structure permits a comparison of the quality of FRC in various sizes, fibre dosage rates, and shapes.

4.1 Environmental structures

FRC has been used in applications where water-tightness is desired such as tunnel linings and liquid storage tanks. Figure 6 shows the typical application for construction of water retention structure using rectangular panels ($h=1.75\text{m}$, $L=1.15\text{m}$, and thickness = 0.1m) with a hybrid rebar-fibre system. Soil and water pressure are imposed as triangularly distributed for various load cases. Post crack tensile strength of 2 MPa is used and by specifying three boundary conditions as fixed and one free to displace and rotate, the member is reduced to a 2-D panel. Numerical simulations show the stress distribution and deflection normal to the plane as well as the yield line mode of failure of the panel (Soranakom, 2008).

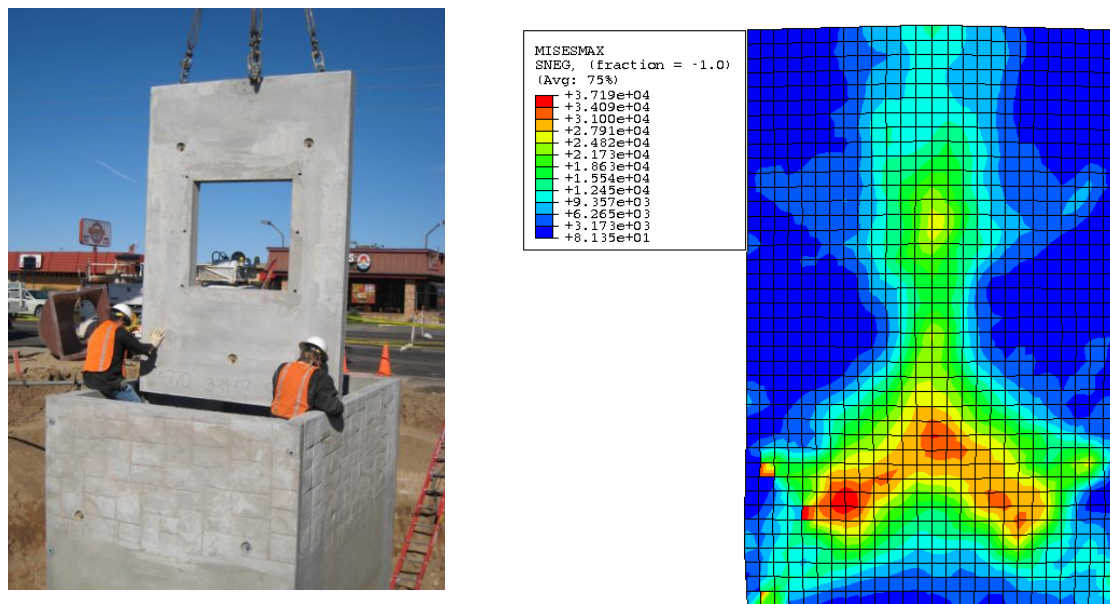


Figure 6: Application of FRC for water retaining structures a) installation of precast panels, b) numerical simulation of deflection distribution in FRC+rebar panel.

4.2 Impact and high speed test development

The high ductility of FRC makes them ideal for use under blast, impact, and dynamic loads. Since the behaviour is dominated by non-linear distributed cracking and frictional dissipative terms such as fibre-pullout, the mechanical properties such as stiffness, ultimate strength, and fracture toughness are highly dependent on the loading rate. In order to promote new applications for FRC several committee reports such as the ACI 544-C Testing Document are being developed. These documents present the latest literature and summarize the available test methods by showcasing potential for use of FRC as a material to withstand high impact loads (Mechtcherine et al., 2011). Uniaxial tension tests on dumbbell-shaped SHCC specimens performed at rates ranging from 10^{-5}s^{-1} to 50s^{-1} . A considerable increase in tensile strain and strain capacity was measured at higher strain rates of $10\text{-}50\text{s}^{-1}$. Microscopic investigation of the fracture surfaces showed that almost no fibre failure and an average pullout length of 2.5mm in the high strain rate test. While the increase in the tensile strength of the fibre was only moderate in the range of strain rates investigated, a considerable increase in bond strength between fibre and matrix was determined.

FRC can absorb the energy through multiple cracking and demonstrates a high tolerance for damage under drop weight tests. Flexural impact tests of FRC reinforced by alkali resistant (AR) glass performed by Silva et al. (2009) report flexural strengths as high as 30 MPa when plate specimens were subjected to potential energies in the range of 7-14 J. The absorbed energy increased with drop height while the ratio of absorbed energy to potential energy decreased with drop height.

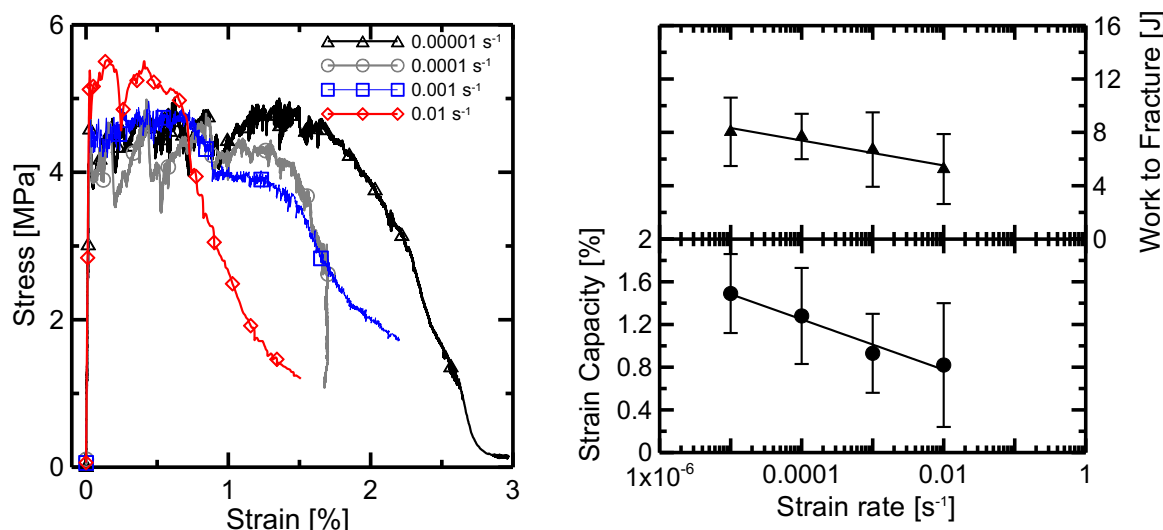


Figure 7: Effect of strain rate on the tensile response of SHCC at low strain rates ($\leq 0.01 \text{ s}^{-1}$)
(a) tensile stress-strain response, (b) effect of strain rate on the strain capacity and work to fracture.

4.3 Corrosion and durability aspects

Concrete is susceptible to degradation through corrosion, alkali-silica reaction, sulphate attack, freeze-thaw damage and other mechanisms that result from the ingress of harmful ions. Concrete durability is therefore, intimately related to the rate at which water and gas are able to penetrate, or leave it, i.e., its water or gas permeability. Dense concrete ($w/c < 0.45$) is nearly impermeable in an uncracked condition. However, permeability is vastly increased by the introduction of cracks and increases with crack width (Wang et al., 1997). Major physical and durability aspects of FRC are addressed in report 544.5R-10 (544.5R-2010).

In the case of SFRC corrosion is much less severe as compared with steel rebar reinforcement. Fibre reinforcement influences the crack development and bifurcation, increased surface roughness of individual cracks and a greater likelihood for crack branching and multiple crack development. Fibre reinforcement may therefore significantly reduce the permeability of cracked concrete. The insufficient knowledge of the deterioration mechanisms caused by fibre corrosion contributes for a conservative design philosophy, which limits the mobilization of the full potential of SFRC (Solgaard, 2010). The consequences of the corrosion in terms of structural performances of a SFRC structures are mitigated if the damage is limited to the formation of micro- and meso-cracks, as self-healing almost restores the concrete integrity. According to Granju et al. (2005), crack width up to 0.1 mm (0.004 in) has no adverse effect in corrosion.

In the area of fire mitigation significant work is underway to extend the exiting ACI reports to address the problems and develop standards (ACI 544.5R, 2010). Along with several other European programs, (Gatti et al., 2004; Nemegeer, 2006) and the *fib* (*fib*, 2010), ACI 544 is collaborating with the *fib* in the development of various chapters for the design of SFRC structures with special consideration for a durability-based design.

5 Conclusions

A summary of recent activities of ACI 544 is presented to address the approaches used in order to promote the use of FRC by codes and specifications development, applications, sustainability and life cycle cost. FRC benefits are discussed in terms of limiting the crack width, and reduction of the high labour cost and time associated with the assembly and placement of rebar during the construction process. It furthermore improves the durability of concrete exposed to corrosive and severe exposure conditions. Many modelling efforts are under way and mechanical and physical properties support the durability and serviceability aspects of this innovative material.

6 References

- ACI 544.5R-10 (2010) *Report on the physical properties and durability of fiber-reinforced concrete*. ACI Committee 544, American Concrete Institute, Farmington Hills, MI.
- ACI 544-7R-2015 (2015) *Report on Indirect Method to Obtain a Stress-Strain diagram for Strain Softening Fiber-Reinforced Concretes (FRCs)*, in press.
- ASTM C1550-12 (2012) *Standard Test Method for Flexural Toughness of Fiber-Reinforced Concrete (Using Centrally Loaded Round Panel)*. ASTM Standard V. 04.02.
- ASTM C1609-10 (2010) *Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam With Third-Point Loading)*. ASTM Standard V. 04.02.
- ASTM C1609-12 (2012) *Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam With Third-Point Loading)*. ASTM Standard V. 04.02.
- Bakhshi, M., & Nasri, V. (2014) Review of International Practice on Critical Aspects of Segmental Tunnel Lining Design. *North American Tunneling: 2014 Proceedings*.
- Bakhshi, M., Barsby, C., Mobasher, B. (2014) "Comparative Evaluation of Early Age Toughness Parameters in Fiber Reinforced Concrete", *Materials and Structures*, 47 (5), 853-872.
- Barros, J. A. O., Cunha, V. M. C. F., Ribero, A. F., and Antunes, J. A. B. (2005) "Post-cracking Behaviour of Steel Fibre Reinforced Concrete," *Materials and Structures*, V. 38, 1, pp. 47-56.
- Bernard E.S. (2000) "Behaviour of Round Steel Fibre Reinforced Concrete Panels under Point Loads," *Materials and Structures*, Vol.33 Apr 2000, pp.181-188.
- Bernard E.S. (2000) "Behaviour of Round Steel Fibre Reinforced Concrete Panels under Point Loads," *Materials and Structures*, Vol.33, pp.181-188.
- Chandler, J. and Neal F. Interim Technical Note 11 (1988) *The design of ground-supported concrete industrial floor slabs*, Cement and Concrete Association, Slough, 16pp.
- De la Fuente, A., Pujadas, P., Blanco, A., Aguado, A. (2012) Experiences in Barcelona with the use of fibres in segmental linings. *Tunnelling and Underground Space Technology*, 27(1), 60-71.

- Destrée, X. (2008) "Free Suspended Elevated Slabs of Steel Fiber-reinforced Concrete: Full-Scale Test Results and Design," *7th International Symposium of Fiber-reinforced Concrete: Design and Applications BEFIB*, R. Gettu, ed., Sept., pp. 941-950.
- Destrée, X. and Mandl, J., (2008) "Steel Fiber Only Reinforced Concrete in Free Suspended Elevated Slabs: Case Studies, Design Assisted by Testing Route, Comparison to the Latest SFRC Standard Documents," *Tailor Made Structure*, J. Walraven and D. Stoeckhorst, eds., Int. FIB 2008 Symposium, Amsterdam, CRC Press, 111 pp.
- Dhir, R.K., Henderson, N.A. (1999) *Specialist Techniques and Materials for Concrete Construction by Technology & Engineering*. 469 pp.
- di Prisco, M., Failla, C., Plizzari, G.A. and Toniolo, G. (2004) "Italian guidelines on SFRC", in *Fiber Reinforced Concrete: from theory to practice*, di Prisco, M. and Plizzari, G.A. Eds., Proc. of the Int. Workshop on advances in Fibre Reinforced Concretes, Bergamo (Italy), September 24-25, pp. 39-72.
- di Prisco, M., Felicetti, R., Plizzari, G. A. PRO 39 (2014) *6th International RILEM Symposium on Fibre-Reinforced Concretes Proc, BEFIB*, Varenna, Italy.
- European Specification for Sprayed Concrete (1996) *European Federation of National Associations of Specialist Contractors and Material Suppliers for the Construction Industry* (EFNARC, 1996).
- fib* (2011) *fib Model Code 2010* Final Draft. Model Code prepared by Special Activity Group 5, Lausanne, September 2011 (Preliminary version).
- Gatti, G.; Fratesi, R.; Signorini, S.; Sonzogno, F. (2004) "Durability of Steel Fibre-Reinforced Concrete Precast Elements: Experiments and Proposal of Design Recommendations", *Proc. 6th RILEM Symposium on Fibre Reinforced Concretes (FRC) – BEFIB 2004*, 20th – 22th September 2004, Varenna, Italy.
- Granju, J.; Balouch, S. (2005) "Corrosion of steel fibre reinforced concrete from the cracks", *Cement and Concrete Research*, V.35, pp. 573-577;
- Grzybowski, M., and Shah, S. P. (1990) "Shrinkage Cracking of Fiber-reinforced Concrete," *ACI Materials Journal*, V. 87, No. 2, Mar.-Apr., pp. 138-148.
- Hansel, D., Guirguis, P. (2011) Steel-fibre-reinforced segmental linings: State-of-the-art and completed projects. *Tunnel*, 30(1).
- Mangat, P.S., Azari, M.M. (1988) "Shrinkage of steel fiber reinforced cement composites," *Materials and Structures*, No. 21, pp163-171.
- Mechtcherine, V. Silva, F., Butler, M., Zhu, D., Mobasher, B., Gao, S. L., Mäder, E. (2011) "Mechanical behaviour of strain-hardening cement-based composites (SHCC) under low and high tensile strain rates", *J. of Adv. Concrete Tech.*, V. 9, 1, 51-62. (2011).
- Mobasher, B., and Destrée, X. (2010) "Design and Construction Aspects of Steel Fiber-Reinforced Concrete Elevated Slabs" *Fiber Reinforced Self-Consolidating Concrete: Research and Applications*, ACI Special Publication 274, Editors: Corina-Maria Aldea and Liberato Ferrara, 2010. SP-274-7, pp. 95-107.
- Mobasher, B., Bakhshi, M., Barsby, C. (2014) "Back-calculation of Residual Tensile Strength of Regular and High Performance Fiber Reinforced Concrete From Flexural Tests" *Construction & Building Materials*, 70 (2014) 243–253.
- Mobasher, B. (2011) *Mechanics of Fiber and Textile Reinforced Cement Composites*, CRC press, Sept, 2011, 480 pp ISBN: 9781439806609.
- Naaman, A. E, and Reinhardt, H. W. (2005) "Proposed Classification of HPFRC Composites Based on Their Tensile Response," *Proc. of the 3rd Int. Conf. on Construction Materials: Performance, Innovations and Structural Implications - ConMat '05 and Mindess Symposium*, pp. 458.

- Nemegeer, D.; Vanbrabant, J.; Stang, H. (2003), "Brite-Euram Program on Steel Fibre Concrete, Durability Corrosion Resistance of Cracked Fibre-Reinforced Concrete", "Test and Design Methods for Steel Fibre Reinforced Concrete-Background and Experiences", B. Schnütgen and L. Vandewalle, eds., RILEM Technical Committee 162, *TDF Workshop, Proceedings* Pro 31, pp. 47-66.
- Reddy, D. (2003) New guide for suspended concrete ground floor slabs, *Concrete*, 2003
- RILEM Committee TC 162-TDF 1995 (2003) "Final Recommendations of RILEM TC 162-TDF: Test and Design Methods for Steel Fibre Reinforced Concrete," *Materials and Structures*, V. 36, No. 36, Oct., pp. 560-567.
- RILEM Final Recommendation TC-162-TDF (2003) "Test and Design Methods for Steel Fibre Reinforced Concrete; σ - ϵ Design Method", *Materials and Structures*, V. 36, pp.560-567.
- RILEM TC162-TDF (2000) "Test and Design Methods for Steel Fibre Reinforced Concrete: Recommendations. Bending test", *Materials and Structures*, 33, 225, 3-5.
- Silfwerbrand, J., (2004) "Design of Steel Fiber-reinforced Concrete Slabs on Grade for Restrained Loading," *Sixth RILEM Symposium on Fiber-Reinforced Concretes (FRC)*, M. Di Prisco, G. A. Plizzari, and F. Roberto, eds., pp. 975-984.
- Silva, F. D. A., Butler, M., Mechtcherine, V., Zhu, D., & Mobasher, B. (2011) Strain rate effect on the tensile behaviour of textile-reinforced concrete under static and dynamic loading. *Materials Science and Engineering: A*, 528(3), 1727-1734.
- Solgaard, A.; Kuter, A.; Edvardsen, C.; Stang, H.; Geiker, M. (2010) "Durability Aspects of Steel Fibre Reinforced Concrete in Civil Infrastructure", *Proc. 2nd International Symposium on Service Life Design for Infrastructure, 4th – 6th October 2010, Delft, The Netherlands*.
- Soranakom, C. (2008) *Multi-Scale Modeling of Fiber and Fabric Reinforced Cement Based Composites*. PhD. Dissertation, Arizona State University.
- Soranakom, C., and B. Mobasher (2010) "Flexural Analysis and Design of Strain Softening Fiber-Reinforced Concrete." ACI Special Publication SP-272, *Antoine E. Naaman Symposium-J*. Editors: Parra-Montesinos & P. Balaguru, 173-187.
- Soranakom, C., and B. Mobasher (2008) "Correlation of Tensile and Flexural Responses of Strain Softening and Strain Hardening Cement Composites." *Cement and Concrete Composites* 30(6): 465-477.
- Soranakom, C., and Mobasher, B. (2009) "Flexural Design of Fiber Reinforced Concrete," *ACI Materials Journal*, September-October, pp. 461-469.
- Soranakom, C., and Mobasher, B. (2008) "Moment-Curvature Response of Strain Softening and Strain Hardening Cement Based Composites," *Cement and Concrete Composites*, Volume 30, Issue 6, pp 465-477.
- The Concrete Society. Technical Report 34, (2003) *Concrete in industrial ground floors – a guide to design and construction*. 3rd Edition, Camberley.
- Vandewalle, L.,(2004) "Test and design methods for steel fibre reinforced concrete proposed by RILEM TC 162-TDF", *Proceeding of the International Workshop "Fiber Reinforced Concrete. From theory to practice"*, Bergamo, September 24-25, 2004, pp. 3-12.
- Vandewalle, L., Dupont D., (2003) "Bending test and interpretation", *International RILEM Workshop on Test and Design Methods for Steel fibre Reinforced Concrete*, Editor(s): B. Schnütgen and L. Vandewalle, RILEM Publications SARL, Pages: 1–13.
- Wang, K., Jansen, D.C., Shah, S.P., Karr, A.F. (1997). "Permeability Study of Cracked Concrete," *Cement and Concrete Research*, Vol. 27, No. 3, pp. 381-93.

FRC design according to the draft Australian bridge code

Stephen Foster

University of New South Wales, Sydney, Australia

Abstract

In January 2014 the Draft for Public Comment Australian Standard for the design of concrete bridges was released (DR AS5100.5); this is the first standard in Australia, and one of the few national standards in the world, to include design procedures for steel-fibre-reinforced concrete in a comprehensive way. This presentation provides the background for the development of the design rules, including on determination of the mechanical properties of the materials, design models for strength and serviceability, considerations with regards to durability and on quality control measures. Validation of the mechanical models adopted for design is also presented.

Keywords

Steel fibres, concrete, design, standards.

1 Introduction

After more than 50 years of research into steel-fibre-reinforced concrete (SFRC) there remain few national standards that deal with the design of SFRC structures and bridges in a comprehensive way. An early adopter of SFRC in standardization is that of the New Zealand Standard NZS 3101 (2006), which 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 (σ -COD) relationship using a prescribed methodology. Models for strength and service design in regards to flexure, shear and axial forces are included.

ACI-318 (2008) introduced a limited allowance for hooked or crimped steel fibres to be used as minimum shear reinforcement in beams and slabs that are not greater than 600 mm in depth and with concrete strengths not exceeding 40 MPa. The dosage of fibres required is typically 60 kg/m³, a volume fraction of 0.75 per cent of the concrete.

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 DafStb Directive for SFRC (2012). Another major source from which design guidance may be found is the *fib* Model Code (2013), which represents much of the current thinking on the topic from Europe and elsewhere.

In January 2014 the Draft for Public Comment Australian Standard for the design of concrete bridges was released (DR AS5100.5); this is the first standard in Australia to include procedures for the design of steel-fibre-reinforced concrete structures. The scope of the draft Standard applies to where steel fibres are used to improve the performance and capacity of reinforced and prestressed concrete members and structures. The design of SFRC at both the ultimate and serviceability limit states is based on establishing the σ -COD relationship at the condition of interest, the background of which is discussed in this paper.

In regards to ductility, DR AS5100.5 stipulates caution when using brittle fibres that rely on a fibre pull-out failure mechanism or where a significant number of fibres may fracture or where fibres result in local crushing of the concrete due to the local forces imposed on the matrix by the fibres. In addition, it is stipulated that steel fibres are not be relied upon in plastic hinge regions unless supported by test data; this is taken directly from the New Zealand Standard NZS 3101 (2006) but needs further investigation. Also excluded from the scope of the Standard is design with strain hardening SFRC, as is depicted in Figure 1. In the drafting of the Standard, the definition of a strain hardening SFRC (Figure 1b) is one of exclusion; that is, it not intended as a performance indicator for strain hardening or ultra-high performance concretes but, rather, a limit as to where the rules designed for strain softening SFRC, as developed for the Standard, can be applied.

This paper provides the background for the development of the design rules for DR AS5100.5 (2013), including determination of core materials properties, design models for strength and serviceability and on quality control deliverables.

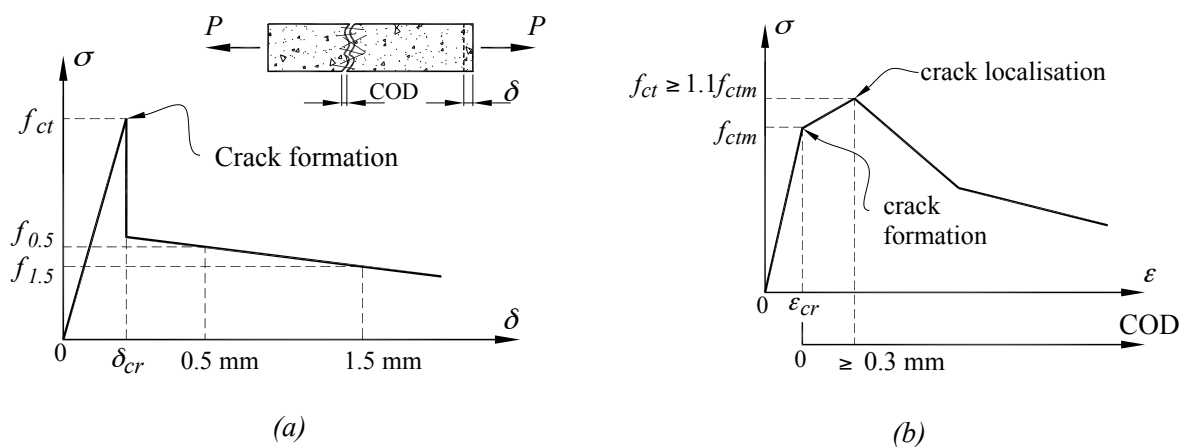


Figure 1: Classification of SFRC according to DR AS5100.5: (a) Strain softening SFRC (b) Strain hardening SFRC.

2 Materials properties and testing

The most fundamental property of SFRC is its post cracking *residual* tensile strength. For this strength a value corresponding to a crack opening displacement (COD) of 1.5 mm in a direct tensile test is adopted (denoted as $f'_{1.5}$, where the prime indicates a characteristic value). To assist designers and suppliers alike, a series standard strength grades are proposed; these are $f'_{1.5}$ values of 0.4 MPa, 0.6 MPa, 0.8 MPa, 1.2 MPa, 1.6 MPa and 2.0 MPa. In the determination of characteristic values for materials properties the Standard, as currently

written, suggests that the population may be treated as normally distributed and a confidence level of 75% shall be used such that 95% of the population exceeds the characteristic value. The assumption of normal distribution requires some consideration; a log normal distribution may be more representative and removes potential for the negative strengths.

The residual direct tensile strength may be determined by a direct tension test (Figure 2) or by a combination of matched direct and indirect testing for a particular mix design. In the latter case, the relationship between the direct and indirect tensile strength is obtained once and, provided that the mix design does not change within set parameters of fibre type and content, water to cementitious material ratio, maximum aggregate particle size and compressive strength, may be used for any project. In this case, the relationship between $f'_{1.5}$ and the flexural strength is determined as:

$$f'_{1.5} = k_1 k_{R,4} f'_{R,4} \quad (1)$$

where k_1 is the boundary (or wall) influence factor and $k_{R,4}$ is a reference factor that provides the relationship between $f'_{R,4}$ and $f'_{1.5}$. In Equation (1), $f'_{R,4}$ is determined from 3-point notched bending test, conducted in accordance with EN 14651 (2007):

$$f_{R,4} = \frac{3F_4 L}{2bh_{sp}^2} \quad (2)$$

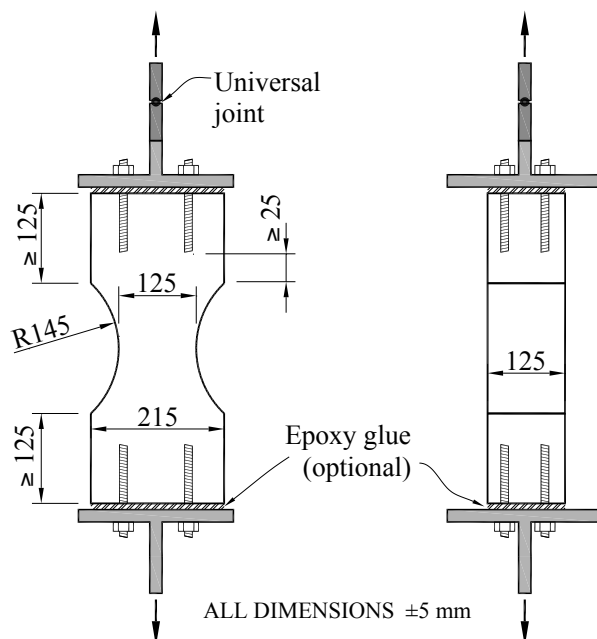


Figure 2: Testing arrangement for determining the direct tensile strength.

where b is the width of the specimen in mm, h_{sp} is the distance between tip of the notch and the top of cross section in mm, L is the span in mm and F_4 the load recorded at a crack mouth opening displacement (CMOD) of 3.5 mm.

The reference factor $k_{R,4}$ is determined as:

$$k_{R,4} = f_{1.5m} / f_{R4,m} \quad (3)$$

where $f_{1.5m}$ is the mean residual tensile strength that corresponds to a crack opening displacement of 1.5 mm and $f_{R4,m}$ is the mean residual flexural tensile strength corresponding to a CMOD of 3.5 mm. With the relationship between the residual (direct) tensile strength and the residual flexural strength established for a given mix, control testing undertaken at the time of construction may be undertaken using the more simple flexural strength testing procedure.

The factor k_1 (in Equation 1) is applied to the direct tension tests to adjust for the wall (boundary) effect and is adapted from Lee et al. (2011) for a square cross-section:

$$k_1 = \frac{1}{0.94 + 0.6l_f/b} \leq 1 \quad (4)$$

where l_f is the length of the fibres.

As no standardised test currently exists for the establishment of direct tension, the literature on fracture was reviewed and the testing arrangement described in Figure 2 was adopted. The arrangement, adapted from that of van Vliet and van Mier (2000), was selected based on: (1) the ease of casting; and (2) stress concentrations that determine a predetermined crack path are reduced, while the failure occurs in a reasonably defined region. While the research of Van Vliet and Van Mier was for plain concrete in tension, Markovic (2004) and Htut (2010) adopted a similar shape for SFRC and showed consistent results. While the end support conditions and boundary rotation effect are important for the brittle response of unreinforced concrete (Van Mier et al., 1995), they become less important for the more ductile post matrix cracking response of SFRC (that is, from the response point after matrix cracking) at the average measured COD of interest (1.5 mm) and where the quasi-brittle response of the matrix has no influence. To this end, one fixed end and one rotating end is used to assist with alignment of the specimen in the testing machine. Further details of the test setup and procedure can be found in Amin et al. (2014). The tension test may be controlled through machine displacements, in which an initial step in COD will occur on cracking, due to recovery of the elastic energy built up in the testing apparatus and specimen, or via external strain control. No displacement or strain rates for the test are specified in the Standard but Amin et al. noted rates of 0.12 mm/min before formation of the dominant crack and 0.2 mm/min after, with additional rate increases as testing progresses.

Alternatively, the characteristic residual tensile strength, $f'_{1.5}$, may be obtained from:

$$f'_{1.5} = 0.4f'_{R,4} - 0.07f'_{R,2} \quad (5)$$

where $f'_{R,2}$ and $f'_{R,4}$ are determined from testing of 3-point notched prisms, in flexure, conducted in accordance with EN 14651 (2007). Equation (5) is adapted from Amin et al. (2014), where the results are compared to test data, as well as the fib Model Code (2013) formulation.

3 Design for strength

3.1 Design for combined bending and axial compression

For strength in bending and axial compression, the simplified stress blocks shown in Figure 3 are adopted. In this case the contribution of the fibres is taken to be plastic with a constant stress 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.

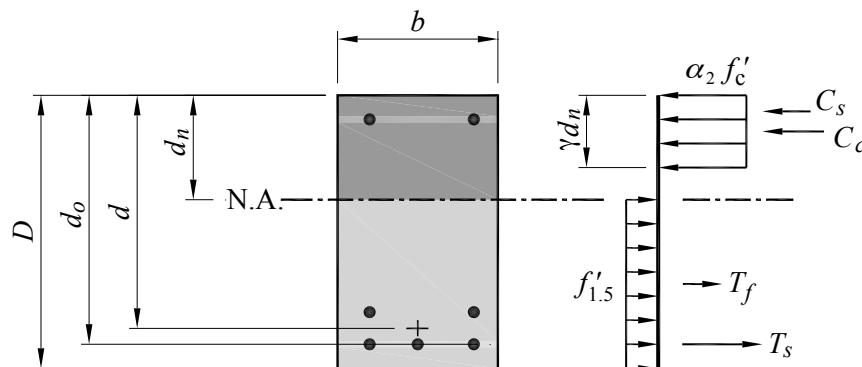


Figure 3: Design for combined bending and axial compression.

3.2 Design for flexural shear

One of the main areas where it is considered that fibres may play a contribution to design practice is in the realm of shear; either as total or partial replacement for steel ligatures. The model adopted for the AS5100.5 draft is based on the simplified modified compression field approach (Bentz et al., 2006) and is adapted from the alternative model presented in the *fib* Model Code (2013). The shear models developed in the *fib* code are based on the “*Level of Approximation*” (LoA) approach (Muttoni and Fernández Ruiz, 2012). With this methodology, design rules are developed based on sound physical-mechanical models with varying levels of simplification. That is, a *Level I* model is based on simplification of the *Level II* model that, in turn, is based on simplification of a *Level III* model, etc. In the approach of the AS5100.5 draft, a *Level I* model is adapted from the *Level II* model presented in the *fib* Model Code (see Figure 4).

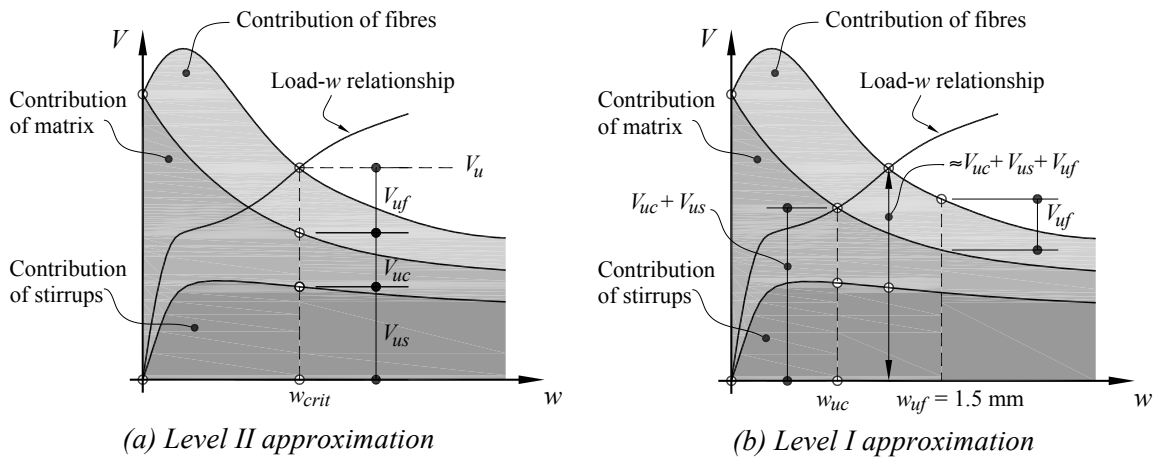


Figure 4: (a) Coupling (LoA I) and (b) decoupling (LoA II) of matrix and fibres components.

It is shown in Foster (2010) that the concrete (V_{uc}) and fibres (V_{uf}) components to the shear strength of a beam are coupled through their common crack width (Figure 4a). In this respect the *fib* model is simplified by assuming a crack width at ultimate for the fibres component of $w_{uf} = 1.5$ mm and the concrete, ligatures and fibres components are decoupled (Figure 4b).

The design shear strength of a beam is then ϕV_u , with $\phi = 0.7$ and:

$$V_u = V_{uc} + V_{uf} + V_{us} \quad (6)$$

where V_{uc} is determined using the simplified modified compression field model and V_{us} is the contributions of the ligatures (Figure 5). The fibre component is:

$$V_{uf} = k_f b_v z f'_{1.5} \cot \theta_{min} \quad (7)$$

where k_f is a factor to account for the variance in fibre distribution and is taken as $k_f = 0.8$ (refer Foster et al., 2013), z is the internal lever arm between the centroids of the flexural tensile and flexural compressive stress resultants, taken as $z = 0.9d_o$, where d_o is as shown in Figure 3, and θ_{min} is the minimum strut angle calculated from:

$$\theta_{min} = 29^\circ + 7000\varepsilon_x \quad (8a)$$

$$w_u = 0.2 + 1000\varepsilon_x = 1.5 \text{ mm} \quad (8b)$$

where ε_x is the longitudinal straining of the web measured at the mid-height of the effective shear section (see *fib* Model Code, 2013).

The minimum strut angle for the fibres component, θ_{min} , is determined as 38 degrees and $\cot(\theta_{min}) = 1.28$. Multiplying the factors gives:

$$V_{uf} = 0.7k_\theta b_v d_o f'_{1.5} \quad (9a)$$

$$k_\theta = \cot \theta_v \leq 1.28 \quad (9b)$$

where θ_v is the angle between the axis of the concrete compression strut and the longitudinal axis of the member (Figure 5).

The model is validated using the data set presented in Foster (2010), with the results presented in Figure 6. The set consists of 180 SFRC reinforced concrete beams that failed in shear (Set A), and 115 (Set B) with the restrictions $a/d \geq 2.5$ and $f_{cm} < 70$ MPa (a is the shear span and d is the effective depth). As the residual tensile strength was not measured in any of the tests in the data set, the tensile strength was calculated using the VEMI Model (Voo and Foster, 2004, Foster et al., 2006). Further details for the data set and the tensile strength model are given in Foster (2010).

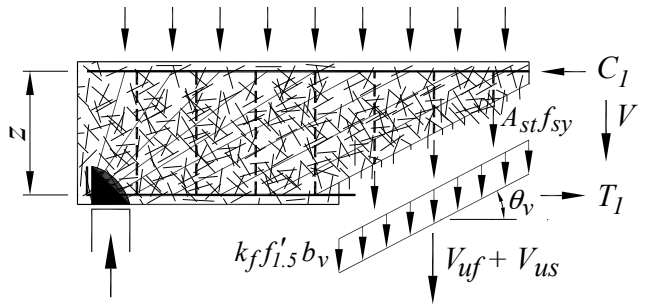


Figure 5: Design for combined bending and axial compression.

Figure 6 shows the AS5100.5 approach is more conservative than that of the *fib* model but with a similar level of variability. The one percentile value in the exponential-to-model ratio for the LoA II model is 0.87, which suggests that the strength reduction factor (ϕ) of 0.7 is sufficiently conservative.

For beams that require shear reinforcement, the minimum contribution provided by the transverse reinforcement (fibres and ligatures) is:

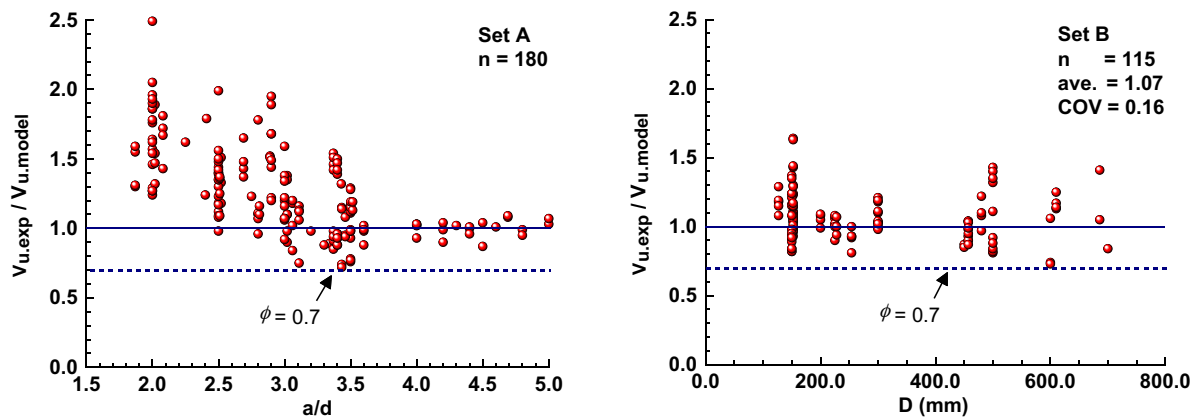
$$(V_{us} + V_{uf})_{\min} = \max(0.1\sqrt{f'_c}, 0.6)b_v d_o \quad (10)$$

For the initial implementation, a conservative approach is adopted such that the maximum contribution of the fibres component to the shear strength, V_{uf} , is limited to the maximum of that given by Equation (10) with V_{us} taken as zero and 30 per cent of V_u .

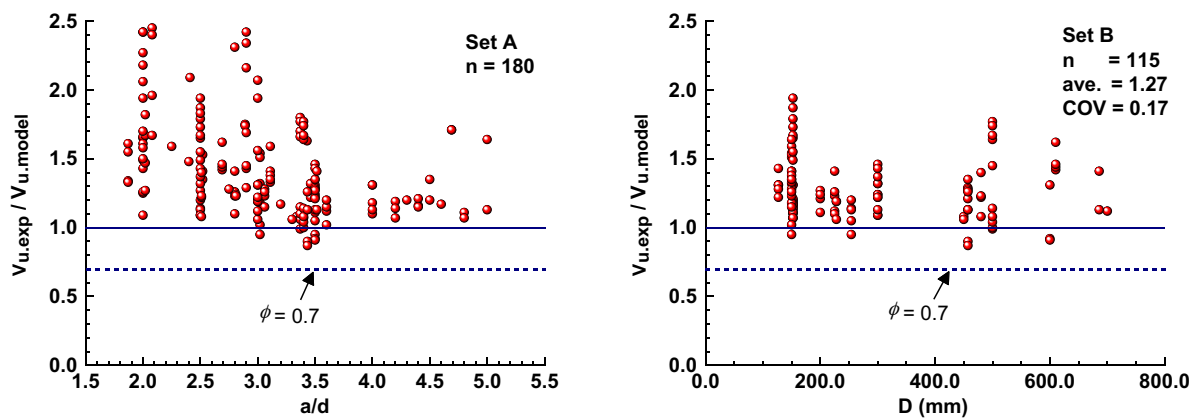
4 Design for service

4.1 Introduction

For service design, steel fibres assist in the control of cracking and deflections. For design it is assumed that a uniform tension is taken by the fibres equivalent to a stress in the concrete of $1.1f'_{l,5}$. In the determination of the minimum longitudinal reinforcement needed for bending, the draft standard ignores the beneficial influence of the fibres; thus localisation of cracking due to the fibres crossing a crack is avoided. In this case, the effect of the fibres is to produce more closely spaced, and finer, cracks. A test on tension stiffening for SFRC undertaken by Amin et al. (2014) is shown in Figure 7. This test was conducted on a 150 mm square section of 1.0 metre length, with an N20 reinforcing bar and 25 kg/m³ of double end-hooked Dramix[®] 5D-65/60-BG fibres. The fibres were 0.9 mm in diameter and 60 mm long. At a COD of 1.5 mm, dog-bone tests gave a mean residual tensile strength of 0.69 MPa. The results of the test, shown in Figure 7, indicate the model adopted to be somewhat conservative.



(a) *fib* Model Code (2013) – Level II Approximation (Foster, 2010)



(b) Draft AS5100.5 (2014) – Level I Approximation

Figure 6: Comparison of *fib* Model Code 2010 Level II approximation with the Draft AS5100.5 Level I approximation for the fibres component (data described in Foster, 2010).

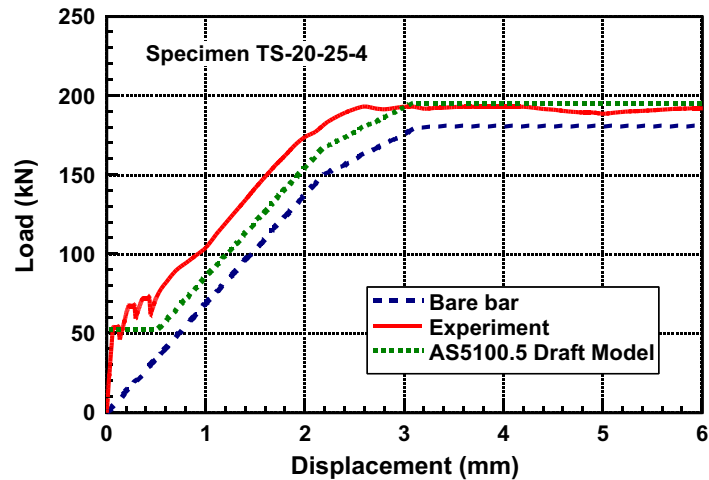
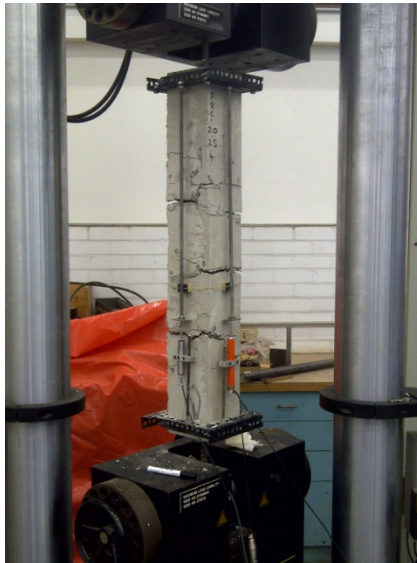


Figure 7: Comparison of tension stiffening test of Amin et al. (2014) with AS3600.5 draft model.

4.2 Crack control

The rules for the control of cracking are derived from NZS3101:Part 2 (2006) and the *fib* Model Code (2013). The minimum amount of longitudinal reinforcement required to obtain controlled crack formation is:

$$A_{st.min} = k k_c k_p f_{ct.ef} - 1.1 f'_{1.5} \frac{A_{ct}}{f_{s.max}} \geq 0.0 \quad (11)$$

where $A_{st.min}$ is the area of reinforcement required within the tensile zone (in mm^2), if $A_{st.min}$ is zero only steel fibres are necessary to control cracking; A_{ct} is the area of concrete on the tensile side of the elastic centroidal axis (in mm^2); $f_{s.max}$ is the maximum stress permitted in the reinforcement immediately after formation of the crack, given in earlier sections of the standard; $f_{ct.ef}$ is the greater of $0.6\sqrt{f_{cm}}$, where f_{cm} is the mean compressive strength of the concrete, and 3.0 MPa. The coefficients k , k_c and k_p are adjustments for shrinkage and temperature, the nature of the stress distribution immediately prior to cracking and for the level of prestress on the section, respectively.

4.3 Deflections

The short-term deflection of an SFRC member is calculated using the model described in Figure 8; the tension stiffening component is taken to be $1.1f'_{1.5}$ through the depth of the tensile zone. This model is then used to determine the effective second moment of area I_{ef} .

Long-term deflections due to shrinkage and creep are calculated separately using the material data specified elsewhere in the Standard, and with the principles of mechanics.

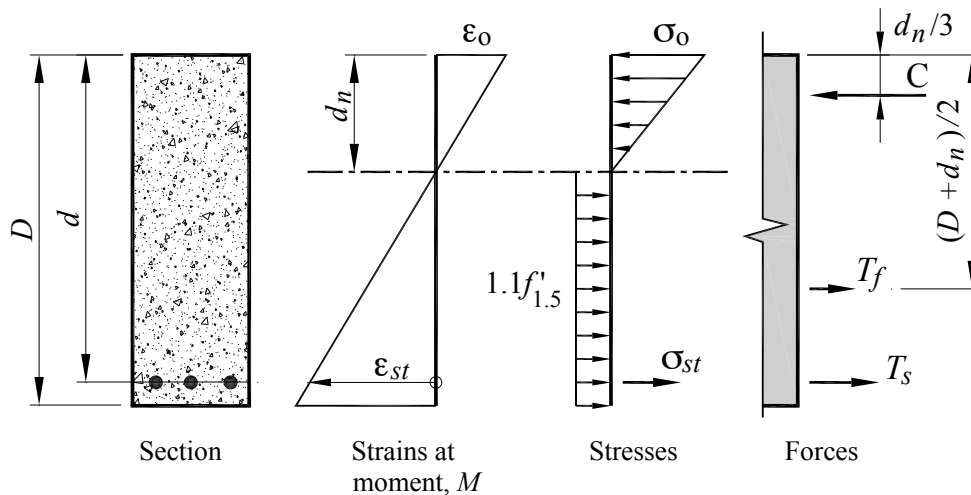


Figure 8: Strain and stress distribution on a cracked section subjected to in-service bending.

5 Quality assurance

While quality assurance is fundamental in the delivery and placement of SFRC, in general, it is paramount in cases where life safety is the essential criteria. While placement and distribution of bars is easily observed before placement of the concrete, this is not the case with SFRC. The principles of reliability need to be applied taking due account of the variability of the placed product and good site control is necessary to ensure the desired distribution of fibres, within the bounds of usual variabilities, and for no cold joint connections.

The draft Standard sets controls on quality at three stages; quality of materials and mixing processes, factory and routine production control and determination on the fibre content and distribution at site. The requirements for production process and finished product inspection are demonstrated in Table 1 and the criteria for acceptance for dosage in Table 2.

6 Conclusions

In January 2014 the Draft for Public Comment Australian Standard for the design of concrete bridges was released (DR AS5100.5, 2014); this is the first standard in Australia, and one of the few national standards in the world, to include design procedures for steel-fibre-reinforced concrete in a comprehensive way. This paper provided some the background for the development of the design rules, including the determination of the materials properties, design models for strength and serviceability and on quality control measures. Validation of the mechanical models for the design of beams for shear was also presented where it was shown that the models provide for sufficiently conservative results. While these models are developed for strain softening SFRC, further work is needed for standardisation with strain hardening and ultra-high performance concretes.