ITAtech Design Guidance For Precast Fibre Reinforced Concrete Segments

ITAtech Activity Group Support

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ITAtech Design Guidance For Precast Fibre Reinforced Concrete Segments

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>> Introduction

This document has been written to assist tunnel designers, contractors and owners in understanding the benefits of and limitations in the use of fibre reinforcement for precast concrete segments for tunnel linings, installed using tunnel boring machines. Guidance is also provided on specifications and testing.

Fibres can be used as reinforcement in precast concrete segmental tunnel linings, either, most commonly, as "fibre only" (as 'Primary' reinforcement) or in combination with conventional (bar) reinforcement - a "combined solution" (as 'Secondary' reinforcement). The state of the art is defined by a large number of reference projects, where fibre reinforced concrete (FRC) segments have been used successfully. Projects using FRC segments report the following benefits of its use:

- Excellent durability;
- Damage due to handling and installation is minimized;
- Performance in the relevant Ultimate and Service Limit States (ULS and SLS) can be reliably demonstrated;
- Reduced waste;
- Overall manufacturing costs are lower than for conventionally reinforced concrete.
- A lower carbon footprint

However, their application in this field has been stifled due to the limited, or even absent, regulatory framework covering this type of product. With the publication of standards specifically dealing with fibre properties, and international design guidelines such as the Model Code 2010, edited by fib, this obstacle has been overcome.

Many research studies and full scale tests on the behaviour of fibre reinforced concrete have been carried out in recent years in various countries. They have greatly contributed to a better characterization of FRC, thereby providing a better understanding of the behaviour of this material and allowing projects to set specific minimum performance requirements.

The aim of this document is to present the common understanding of designers, manufacturers and users of fibre reinforced concrete segments of what constitutes good practice in this field of engineering. This is the first edition of what is intended to be a "live document". ITAtech welcomes all feedback on this document and has plans to keep this document up to date as well as publishing guidance on production aspects of FRC.

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Aspect ratio:
The ratio of length to another dimension – e.g. segment thickness or equivalent fibre diameter

CMOD:
Crack Mouth Opening Displacement which is the linear displacement measured by a transducer – e.g. installed on a beam in a test according to EN 14651

Ductility:
This is a measure of a material's ability to undergo appreciable plastic deformation under tensile stresses before rupture. The ductility of a structure is defined in the same way as a measure of the ability to undergo appreciable deformation under tensile forces before failure.

Energy absorption:
This is the area under a load-deflection, moment-rotation, or stress-strain curve. This is the ability of FRC to absorb energy beyond the first crack in the matrix. This is not a parameter that is used directly in design of segmental linings, unlike sprayed concrete linings for rock tunnels.

LOP:
Limit of Proportionality – the stress which is assumed to act in an uncracked mid span section of flexural beam test (e.g. see EN 14651). This is the point at which the load-deflection curve departs from the initial linear response.

Modulus of rupture: The theoretical maximum tensile stress in a flexural beam test before fracture, based on elastic theory (e.g. see ASTM 1609).

PFRCS: Precast Fibre Reinforced Concrete Segments

Residual strength:
This is the flexural tensile strength exhibited by a fibre reinforced concrete after cracking. In tests, the residual strength is usually defined at a certain deflection or CMOD.

Strain hardening:
In the context of tensile tests, hardening means that the post-cracking strength is higher than the strength at first cracking – see also strain softening.

Strain softening:
In the context of tensile tests, softening means that the post-cracking strength is lower than the strength at first cracking. Although this is also described as strain softening, strictly speaking, the tests are measuring strength vs deflection, i.e. deflection softening (see 1.3.2).

Strength at first crack: see LOP

Toughness:
This is the ability of a material to absorb energy and plastically deform without rupturing. This indicates the ability to resist internal crack propagation.

>> Notation

b	Breadth
CMOD	Crack Mouth Opening Displacement
d	Diameter of fibre
E	Elastic modulus
E_{cm}	FRC Secant Compression Modulus
E_m	Concrete (Matrix) Tangent Compression Modulus
E_f	Fibre Elastic Modulus
E_t	FRC Secant Tension Modulus
f_{cm}	Concrete Mean Compressive Cylinder Strength
f_{ck}	Concrete Characteristic Compressive Cylinder Strength
f_{ctm}	Concrete Mean Tensile Strength
f_{ctk}	Concrete Characteristic Tensile Strength
f_{cd}	Concrete Design Compressive Strength
f_L	Limit of Proportionality stress of FRC beam
flim	Design tensile strength - see Figure 4
f_{Ri}	Residual flexural Strength of FRC beam
f_{Rim}	Mean Residual strength of FRC beam
f_{Rik}	Characteristic Residual Strength of FRC beam
f_{Fts}	FRC Tensile Strength at SLS (Model Code 2010)
f_{Ftu}	FRC Tensile Strength at ULS (Model Code 2010)
f_y	Tensile yield strength
FEA	Finite Element Analysis
fib	Federation International du Beton
FRC	Fibre Reinforced Concrete

>> Notation

h	Height
k_w	Coefficient - statistical
K	Coefficient
l	Fibre length
Lc	Characteristic length
LE	Linear Elastic
LOP	Limit of Proportionality
MOR	Modulus of Rupture
Mu	Ultimate moment
P	Pressure (load)
R	Radius
RILEM	Reunion Internationale des Laboratoire d' Essais Materiaux
SFRC	Steel fibre reinforced concrete
SLS	Serviceability Limit State (limit of the service requirements)
S_n	Standard Deviation
ULS	Ultimate Limit State (limit associated with structural failure)
V_f	fibre volume content
V_x	Coefficient of variation
w_k	Characteristic crack width
γ	Material factor
δ	Deflection
ε	Strain
θ	Angle
σ	Stress
τ	Shear strength

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ITAtech gratefully acknowledges the contributions of the members of the PFRCS group and others, notably: Figure 7 Reprinted, with permission, from C1609/C1609M-12 Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam With Third-Point Loading), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428, www.astm.org

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Traditionally, concrete segments for shield excavated tunnels have used conventional bar reinforcement. However, there is a growing tendency to consider fibre reinforcement as structural reinforcement, mainly in order to benefit from the significant cost and time savings which can be achieved but sometimes also because of the enhanced durability that fibres offer.

A significant number of international reference projects have demonstrated that a high quality tunnel lining can be achieved using fibre reinforced segments. However, the previous lack of a common design standards or guidelines tended to discourage some clients from accepting design proposals based on fibre reinforced concrete.

This document is based mainly on experience gathered during the use of Eurocodes and the international fib Model Code. However, this merely reflects the engineering background of the authors. The principles described here can be applied to other codes such as ACI318. Fibre reinforcement has been used successfully throughout the world.

>> Objective of the guidance document

The aim of this document is to provide comprehensive guidance on the use of fibre reinforcement for tunnel lining segments. Section 1 describes this composite material and how it behaves. Section 2 is intended to illustrate general concepts in design of Fibre Reinforced Concrete (FRC), including structural design, design codes, numerical modelling and fire protection. Section 3 provides guidance on specifications and testing. Extracts of fib's Model Code can found in Appendix A while Appendix B explains how to determine design strengths for residual flexural tensile strength. Full scale tests can also be used in design as illustrated in Appendix C. Appendix D contains some examples of previous successful projects. Appendix E explains the European CE marking system for fibres. Appendix F contains an example of the cost comparison with conventional bar reinforcement which indicates that the cost savings could be of the order of 40%. Obviously the savings will vary depending on the requirements of each project.

It is intended that a future document will cover the topics of segment production, quality control (tests and their frequencies), handling, storage, transportation and installation.

1.1 DESIGN STANDARDS

At the time of writing, the design of fibre reinforced concrete (FRC) is not covered by international structural codes like the Eurocode 2 or ACI 318. Therefore, the design rules applied in projects are taken from a number of sources. From an European perspective, it is expected that FRC will be introduced in the Eurocode suite of standards and that the fib Model Code (fib MC2010) will be used as a basis for this Eurocode. The parameters for the fibre reinforced concrete are defined by associated test standards, such as EN 14651.

Due to the publication of a number of guidelines and reports and the significant amount of testing over the last decades, the general behaviour of FRC is sufficiently well understood to allow production of designs to an acceptable "reliability level", in Eurocode terminology. In combination with the appropriate testing, the reliability level prescribed by concrete design standards can be achieved, even though FRC design itself is not standardised in national design codes yet. Below is a selection of applicable design codes and references which are commonly used and accepted. A list of testing standards can be found in section 3.

References relevant to FRC design:

I- RILEM TC 162-TDF, Test and design methods for SFRC. Materials and Structures, Vol. 36, 2003

II- FIB (2013) - International Federation for Structural Concrete. fib Model Code for Concrete Structures 2010. Berlin: Verlag Ernst & Sohn, 2013

III- Deutscher Ausschuss für Stahlbeton, DAfStB – Richtlinie Stahlfaserbeton, 2012

IV- ÖVBB Richtlinie Faserbeton, Ausgabe Juli 2010: Guideline on the use of fibre reinforced concrete, Österreichsche Vereinigung für Beton- und Bautechnik (2010)

V- CNR-DT 204/2006 (2006), „Guide for the Design and Construction of Fiber-Reinforced Concrete Structures", design recommendation. Advisory Committee on Technical Recommendations for Construction, Rome

VI- BTS Specification for Tunnelling, 3rd Edition, 2010

VII- Recommendations for the design, production and installation of segmental rings 07.2013 Published by the DAUB (German Tunnelling Committee) working group «Lining Segment Design»

VIII- AFTES (2013) "Design, dimensioning and execution of precast steel fibre reinforced concrete arch segments", recommendations of AFTES No. GT38R1A1, Tunnels et espaces souterrain, No. 238 July/ August 2013

IX- Concrete Society Report TR 63 Guidance for the Design of steel fibre reinforced concrete

X- Concrete Society Report TR 65, Guidance on Macro-synthetic fibre reinforced concrete

XI- ACI 544.5R-10 (2010) Report on the Physical Properties and Durability of fiber Reinforced Concrete, ACI, March

1.2 FIBRES TYPES AND PROPERTIES

Fibres are concrete additives from a concrete technology point of view. Fibres are used to improve concrete properties. As the applications have diversified so have the materials used for fibres and their shapes. Fibres have become a standard reinforcement method for industrial floors, pavements, housing applications, sprayed concrete linings and a variety of precast applications. Over the past decades, steel fibres have developed as a replacement of the reinforcing steel bars in precast segments. Macro synthetic fibres have been developed too as structural reinforcement for segments. A significant number of research projects have accompanied this shift of fibres into more structural applications (e.g. see Concrete Society TR 63 and TR 65 for an overview, as well as ITA WG2's forthcoming report (www.ita-aites.org)).

In ACI 544.5R-10, Report on the Physical Properties and Durability of Fiber-Reinforced Concrete, there is a comprehensive resume of the mechanical properties of the fibres used for concrete reinforcement. It is important to assess the effect of a fibre on the performance of the final FRC and not just the properties of the fibres in isolation.

The following general observations can be made:

- The materials should satisfy the following requirements

  - The fibre material must be suitably resistant to the alkaline environment of the concrete.
  - The material properties of the concrete should not suffer significant negative effects from the fibres used. This requirement applies for the properties of both the fresh concrete (workability, air content, etc.) and hardened concrete. With respect to the properties of the hardened concrete, there should be no negative change to the compressive strength and splitting tensile strength, static modulus of elasticity, the creep and shrinkage behaviour, the bond of a reinforcement as well as the durability (resistance to carbonation, frost resistance, water penetration depth, etc.).
  - The fibre material must not deteriorate under normal storage conditions.
  - The fibre material must be suitable for the intended temperature range in use and for the required fire resistance.
- The fibres must satisfy the existing requirements for environmental compatibility and being harmless in physiological terms.
- The anchor system should be optimised (e.g. hooks at the end, embossing, crimpled shapes or other systems

to ensure the fibre is anchored in the concrete matrix)

- Concrete mixing must:
  - use an automatic dosing system for the fibres and
  - mix the fibres in the concrete to obtain a uniform distribution
- The performance of fibre reinforcement in concrete relies on ensuring both an adequate dosage of fibres and an even distribution. Trials to check the distribution and orientation of the fibres should be made during the mix design phase. This is particularly important, if the aspect ratio for steel fibres (l/D) is greater than 65.

1.3 BEHAVIOUR OF FIBRE REINFORCED CONCRETE

To assess whether or not FRC is suitable for a project, it is important to understand its general mechanical performance and its limitations compared to conventional bar reinforced concrete. The main components that need to be examined are:

- The concrete matrix strength
- The concrete matrix durability class
- The type and the quantity of fibres (including the fibre properties)
- The fibre-matrix interface properties, like post crack residual strength

Fibres can be used as reinforcement on their own or in combination with conventional steel bar reinforcement. Fibres work in tension across cracks in concrete and thus provide residual flexural capacity for cracked concrete. At typical dosages, the enhancement in flexural capacity is modest but this is often adequate since tunnel segments are generally designed to minimise the bending moments in the segments.

Fibres may be used for other reasons too, such as:

- to enhance the performance of the segment during handling and installation;
- or to enhance the performance of segments reinforced with conventional reinforcing bars in response to extraordinary load cases;
- or to reduce average crack widths in bending.

In comparison to bar reinforcement, the characteristics of fibre reinforcement are that:

- the fibres are distributed throughout a cross-section, whereas reinforcement bars are placed in specific locations and require concrete cover to the faces of the segment;
- the fibres are small, relatively short and closely spaced, whereas the reinforcement bars are larger, continuous and not as closely placed;
- it is not generally possible to achieve the same weight per cubic metre of reinforcement with fibres as with reinforcing bars.

Regarding the mechanical performance of FRC, the following key effects are:

a. Fibre reinforcement provides resistance by being “pulled out” of the cracked concrete. The anchorage (including bond in shear and/or adhesion to the concrete matrix at the fibre interface) is of significant importance to the post-crack behaviour. This is a quite different concept when compared to conventional bar reinforcement.

b. A post cracking performance of FRC is provided by the presence of fibres.

c. Usually, the composite material exhibits strain-softening behaviour in direct tension, although strain-hardening can also be achieved. The fibres in FRC promote crack-bridging, transferring stress across the cracks to the concrete matrix. This explains the improved post crack stress-strain curve, compared to plain concrete.

d. Fibres contribute to controlling concrete cracking.

e. The presence of fibres does not influence the mechanical performance of “uncracked” concrete[1] (prior to the first crack in testing).

The anchorage depends on:

- the shape of the anchorage
- the friction between the fibre and concrete (quality of fibre/crack interface and orientation),
- the density of the concrete matrix
- the tensile strength of the fibre

1.3.1 Behaviour in compression

Compared to a reference (unreinforced) concrete, FRC (at moderate dosages of fibre, i.e. less than 0,6% per volume - which equates to 45kg/m^3 of steel fibres or 6 kg/m^3 of macro synthetic fibres) tends not to exhibit any particularly different behaviour in compression. Therefore, the stress-strain ratios given in the respective concrete design codes can be used without modifications.

1.3.2 Behaviour in tension

The direct tensile capacity of reinforced concrete is characterised by an almost linear elastic behaviour up to the point of tensile failure (cracking) of the concrete matrix. This failure occurs at relatively low strains, so that fibre reinforcement typically does not enhance the tensile resistance up to this point. In consequence, the peak characteristic tensile strength of fibre reinforced concrete can be taken to be at least equal to the direct tensile strength of plain concrete.

The behaviour of fibre reinforced concrete may be classified as strain softening or strain-hardening (i.e. a post cracking strength which is larger than the strength at first cracking). For strain softening materials, localized individual cracks form causing the stress which it can carry to decrease with increasing deformation. Strain-hardening FRC is sometimes called high performance fibre reinforced cement composite (HPFRCC) (Lofgren 2005). It is possible to

[1] At dosages < 45 kg/m^3 for steel fibres or 6 kg/m^3 of macro synthetic fibres

distinguish three different stages:

- Linear elastic stage
- Micro-cracking stage
- Macro-cracking stage

In the absence of more detailed information derived from testing, it can be assumed that at moderate fibre dosages, FRC is likely to exhibit strain-softening behaviour in tension.

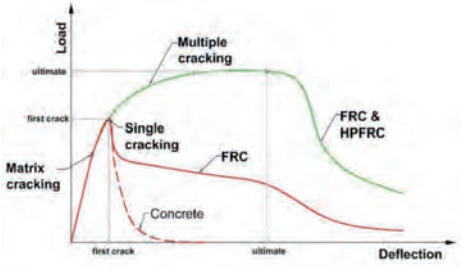

Figure 1: Classification of tensile behaviour of cement-based materials (by Psomas after Lofgren 2005)

1.3.3 Behaviour in flexure

The typical purpose of fibre reinforcement is to impart flexural capacity and therefore ductility into cracked concrete. The failure mode in flexure is a plastic fibre pull out process as opposed to elasto-plastic deformation observed in bar reinforced concrete and the reinforcement material itself. Compared to "plain" concrete, fibre reinforced concrete retains flexural residual capacity in the post-crack region, thereby avoiding brittle failure. Flexural crack widths have been found to be reduced through the inclusion of fibres in combination with conventional reinforcement or when used at sufficient dosages to produce a strain-hardening behaviour in flexure.

The fibres act in a bridging mechanism across cracks, resulting in the mobilisation of post-cracking tensile bridging stress and energy dissipation. The load sharing contribution from fibre bridging appears gradually, beginning after the crack opening widths exceed approximately 0.05 mm.

It is important to note that this "ductility" is affected by inter alia the volume of fibres, the number of fibres ("fibre count"), the aspect ratio (for steel fibres), the anchorage mechanism and by the relative strength of the fibre and its bond the concrete matrix (which must be at least one order of magnitude higher for steel fibres). The fibre volume needs to exceed a critical value in order to achieve deflection hardening. This critical value is different for each fibre and concrete and it can be confirmed by testing. Hence, the design for flexure of FRC requires careful consideration of stresses and the associated strain states.

1.3.4 Elastic modulus

Compared to a similar unreinforced concrete, FRC tends not to exhibit any particularly different behaviour in compression and tension up to the Modulus of Rupture, MOR, or the limit of proportionality, LOP. Therefore, the equations for the elastic modulus, E, given in the normal design standards can be used without further modification. Similarly, the Poisson's ratio and shear modulus of the concrete matrix are not changed by the addition of fibres.

1.3.5 Long-term properties of FRC

When considering creep, it is helpful to distinguish between the potential effects on the concrete matrix and the effect on the fibre reinforcement. As the mechanical properties of the concrete matrix are not modified by the presence of fibres in compression, it appears reasonable to assume that creep behaviour in compression is comparable to conventional concrete. Like all reinforced concrete, FRC does creep in tension as well as in compression. The creep behaviour depends on the long-term stress level and the properties of the fibre. For concrete in flexure, the long-term cracked behaviour depends on the properties of the fibre and the concrete. Further guidance on the performance of the selected fibre in creep should be obtained from the manufacturer. As mentioned above, fibres provide ductile flexural resistance by being gradually pulled out of the concrete. This effect is highly dependent on the strength and hardness of the bond between the cement matrix and the individual fibre. Some concretes exhibit significant long term strength gain. Due to this hardening of the concrete over months and years, this bond could increase during the design life of the segment. It is possible that the bond strength could reach a value which prevents this "pulling out" effect, instead leading to high strains in the individual fibres. This can cause them to "snap", resulting in failure below the values required for ductility, even though they passed these required values at tests at 28 days.

Therefore a suitable dosage of a fibre, which is appropriate for the long term strength of the concrete, must be selected to ensure ductile behaviour of the composite material. Alternatively, the strength gain of the concrete should be restricted to a value which is compatible with the selected fibre. This ensures that the fibres fail in the intended manner by "pulling out".

This effect can be relevant to segmental lining designs which experience cyclic loading or in other special applications where additional flexural strength has to be mobilised late in the design life.

1.3.6 Effects of fibres on crack widths

Flexural crack widths have been found to be reduced through the inclusion of fibres in combination with conventional reinforcement or when used at sufficient dosage rates to produce a strain-hardening behaviour in flexural tension. In a deflection-softening FRC the first crack to occur in a segment is likely to become enlarged as a result of strain localization and this

will therefore become the dominant crack, if deformation continues. In this circumstance, fibres have no effect on average crack width.

1.3.7 Behaviour under dynamic loads

It is well established that fibre reinforcement improves the energy absorption capacity of concrete by enhancing its post-peak load transfer capability. This corresponds to increased toughness and therefore it provides an effective way of improving the resistance of concrete to impact. The degree of this improvement is dependent upon the fibre type (material, length, shape, aspect ratio, etc) and thus a comprehensive review of the FRC performance must be carried out prior to its selection for any application that experiences dynamic loading.

It should be noted that FRC is successfully used in a number of non-tunnel applications under dynamic loads, e.g. railway / tram track bed (Ridout 2009). However, the suitability depends on the strain level as well as on the material. This document cannot give any “hard and fast” rules on the applicability of FRC for dynamic loads.

Fibres are known to provide confinement to concrete in the event of cracking due to seismic loading. The degree of confinement is believed to depend on the dosage rate and characteristics of the fibres used. This effect has been found to be particularly advantageous when FRC is used together with conventional reinforcing bars. No guidelines are available at present to quantify this effect and thus additional testing may be required for a project to exploit this property of FRC.

The behaviour of FRC under dynamic loads is subject of ongoing research with very encouraging results. For example, High Performance Fibre Reinforced Concrete (HPFRC) appears to exhibit strain hardening behaviour in bending and quite resilient behaviour under dynamic loading.

2.1 GENERAL REMARKS

A common approach is to use design parameters based on characteristic material parameters for "plain" concrete in the first iteration of the design and allow for reinforced concrete factors of safety in order to obtain "design" material parameters. This is a conservative approach and assumes that a minimum ductility level must be demonstrated in structural testing. For example, the Model Code 2010 (fib MC2010) suggests the following minimum requirements:

- The flexural resistance of the FRC at the strain level used for SLS design *(at CMOD=0.5mm, f_{R1k})* must be greater than 0.4 times the flexural resistance at the limit of proportionality *(first crack, f_{Lk})*
  $f_{R1k}/f_{Lk} > 0.4$

and

- The flexural resistance of the FRC at the strain level used for ULS design *(at CMOD=2.5mm, f_{R3k})* must be greater than 0.5 times the flexural resistance at the strain level used for SLS design *(at CMOD=0.5mm, f_{R1k})*
  $f_{R3k}/f_{R1k} > 0.5$

If these requirements are both met, fibres can be used to replace bar reinforcement.

A better approach is to use design parameters based on characteristic material parameters for FRC (e.g. see Appendix B). When using a Limit State design approach, the characteristic flexural strength is reduced by an appropriate material performance reduction factor (which is known as a partial safety factor or strength reduction factor) and then the segment capacity in bending is calculated. This will then be compared to a factored load action. The following sections will focus on design using this approach since most concrete design codes use a similar approach (e.g. Model Code 2010, ACI318 or Eurocode 2). Obviously the requirement to achieve a minimum ductility still applies.

The design checks required for fibre reinforced concrete are the same as those required for a conventionally reinforced segmental lining. The design checks must address the following particular issues as a minimum:

I- Capacity of the segments for bending and compression

II- Detailed checks pertaining to the radial joints

III- Detailed checks of the segments for stresses resulting from ram forces

IV- Transportation and handling load cases

V- Further detailed checks, e.g. gasket groove

Issues ii and iii in the list above govern the design in the majority of cases. As noted earlier, designers aim to limit the bending in the segments. This is partly achieved by controlling the aspect ratio, which is defined below with an example in Figure 2. The experience from numerous projects suggests that an aspect ratio of less than 10 is suitable for segments made of FRC alone. Higher aspect ratios may be possible depending on the methods of handling, transportation, installation, TBM driving and ground loads.

$$\lambda = \frac{\textit{mean developed segment length}}{\textit{segment thickness}}$$

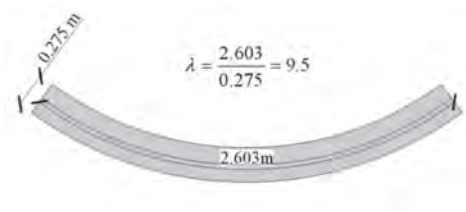

Figure 2: An example of a segment aspect ratio λ.

2.2 ULS DESIGN CONCEPT

2.2.1 General

An appropriately specified quality and quantity of fibres will result in sufficient ductility to prevent brittle behaviour in failure modes relevant to segment design. Consequently fibre reinforced concrete segments can be designed to a similar level of safety as a conventional (bar reinforced) segment, using the same partial material factors as for reinforced concrete. Although this is discussed in more depth in Appendix B, in simple terms, the partial material factor of 1.5 in Eurocode terminology (or a strength reduction factor of 0.65 in ACI terminology) can be used for determining FRC design strengths.

This approach assumes implicitly that the characteristic material parameters used in the design have been specified and tested to the same level of reliability as conventionally reinforced concrete under comparable conditions. Material characterisation through testing is required as part of the verification of the design. It is the designer's responsibility to ensure this level of confidence is achieved by specifying an appropriate level of material testing (see section 3). The design method should be consistent with the test method.

The required checks for permanent and transient design situations can be generalised into

I- Design for flexure

II- Design for shear

III- Design for tension

IV- Design for compression

Since the characterisation of the material is based on testing (rather than on prescription), there is a need to relate the load-displacement or load-crack opening information to a stress strain curve, which is used in the design. This process is described in different ways in a number of guidance publications. There are two basic approaches for the design of FRC members

- Approach A: The σ-ε (stress-strain) method, where the load–deflection or load–CMOD (crack mouth opening displacement) relationship is deduced from testing. The strength is linked to the

load at predefined CMODs (or deflections), or to the area beneath the curve up to a predefined deflection; this in turn is used to derive the residual or equivalent flexural strengths. The CMODs are transformed into strains and the residual (or equivalent) flexural strengths are translated into direct tensile strengths (see di Prisco et al 2013).

- Approach B: The σ-w (stress-crack width) method, where tensile testing yields a load–crack width relationship which is mainly used directly in formulations based on fracture mechanics The principles behind this approach are summarised in section 2.4.2.

The Approach A is more common in designing FRC members. Approach B requires a more comprehensive characterisation of the material based on direct tensile testing.

2.2.2 Design for flexure

The design of FRC in flexure is covered in a number of detailed publications (see section 1.1). Most, if not all, references recommend using a stress block to model the effect of fibres in flexure (e.g. see Appendix A). The resistance of a FRC cross-section to a combined axial force and bending moment can be considered in a Moment – Axial thrust interaction diagram[2] The equilibrium of forces and bending moments is determined in the same way as for conventional bar reinforcement. Figure 3 shows an illustration of the contribution of fibre reinforcement in comparison to bar reinforcement, following the fib Model Code approach. Similar results can be obtained using other design methods such as ACI 544.FR (Bakhshi & Nasri 2014). The figure shows that fibres provide a significant improvement in bending capacity in the most critical area. In general, segments are designed to work mainly in compression, with low bending moments. Also it should be remembered that tunnel linings embedded in the ground are highly redundant structures which tend not to suffer high, local loads in the long term.

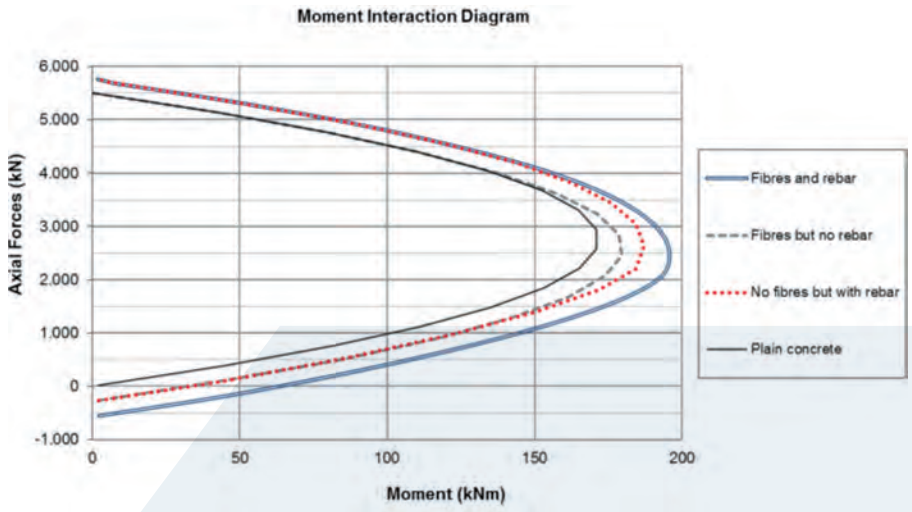

Figure 3: Moment – Axial thrust interaction diagram showing various forms of reinforcement (for 250 mm of C40/50 5c concrete with 8/8 mm 150/150 mm c/c mesh)[3]

2.2.3 Design for shear

The main action for shear transfer across a crack in "plain" concrete is attributed to aggregate interlock and friction at the crack face. For fibre-reinforced concrete, at normal fibre dosages, as soon as the matrix cracks, the fibres are activated and start to be pulled out, providing additional resistance and contributing to the control of shear cracking.

Various design guidelines permit the enhancement of the shear capacity due to the fibres in the case where conventional bar reinforcement exists. However, in the ideal case, fibres would replace all of the bars in a segmental lining. Little guidance exists on how to calculate the shear capacity of FRC in this case. Coccia et al (2015) have rightly pointed out that fib MC2010 offers guidance based on limiting the principal tensile stress to less than the design tensile strength and that this is difficult to evaluate when bending and axial force are present. The authors propose the use of a reduced design value for the residual flexural strength to take this limitation into account. This value can then be used to create a modified Moment – Axial thrust interaction diagram. They go further, to propose a simplified check, wherein a chart can be used to determine whether or not the moment capacity is reduced by more than 10% - see Figure 4. If the capacity is affected by less than 10%, they propose that the effect of shear can be ignored.

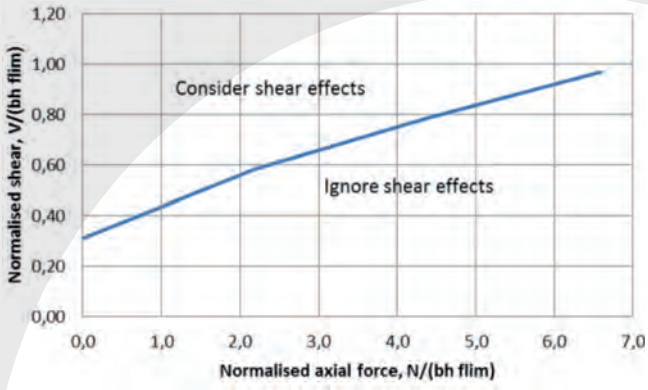

Figure 4: Simplified shear check after Coccia et al (2015). Where V is the shear force, N is the axial force, b is breadth, h is depth and flim is the design tensile strength.

[2] Currently there is no established method of dealing with concurrent tension and bending for FRC, although in principle this is possible through closed-form solutions following conventional stress block section analysis.

[3] This figure is based on Eurocode 2 and the Model Code 2010 with a rectangular stress block in the tension zone.

2.2.4 Design for tension

Design using the tensile capacity of FRC is normally required in areas like those subject to peak stresses from bursting caused by ram forces or close to radial joints. The associated failure modes occur at very small strains, hence the effect of fibres in this design case should not be explicitly considered, unless substantiated by appropriate full scale tests. In the absence of results from such tests, the associated design checks should be based on the design tensile capacity of the concrete. As mentioned in section 1.3.2, the tensile strength is not altered by the fibres so the established relationships for estimating the tensile strength of plain concrete should be used.

Should the tensile capacity of the concrete prove insufficient to demonstrate compliance with the design code requirements in the respective ULS, appropriate resistance must be provided by additional bar reinforcement, or by quantifying the effects of fibres through full scale testing. The capacity of joints is often much higher than design codes would predict so full scale testing or empirical guidelines, developed by design companies, are often used in practice.

For tunnel linings that are designed for direct tension design situations (e.g. pressurized water tunnels or special load cases), deflection hardening in direct tension is required.

2.2.5 Design for compression

For practical dosages, the effect of fibre reinforcement on the performance of concrete in compression can be ignored. Generally, the presence of fibres improves the ductility of high strength concrete and leads therefore to a more benign behaviour near the compressive limit. It is anticipated that this effect will reduce the spalling tendency of FRC and generally improve behaviour at the joints.

2.3 SLS DESIGN CONCEPT

2.3.1 Crack width control

While FRC normally performs better than bar reinforced concrete in terms of durability, target crack widths are still selected to prevent fibre corrosion, to comply with water tightness criteria, to ensure concrete matrix durability, or simply to achieve the surface finish specified by the client. For corrosion protection of steel fibres, a crack width limit of 0.15mm to 0.20mm can be considered as best practice, depending on the aggressivity of the environment (AFTES 2013). Alternatively, it may be more convenient to allow for steel fibre degradation in the outermost FRC layer and to ignore this layer in the design checks.

Experience has shown that fibres control early age shrinkage cracking well. However, drying shrinkage cracking does not usually occur in segment manufacture.

In flexure, generally, crack widths can only be reduced by achieving deflection hardening material behaviour in the FRC. In this case, the crack spacing for the “fibre only” (Primary) case, and from that the resulting crack width, can be estimated based on the strains occurring in the particular design situation with empirical relations given, for example, in the DAfStB Richtlinie, or be determined experimentally from large scale testing. It is strongly recommended to verify any design for crack control to an explicit value by a testing schedule.

Alternatively, it is also possible to use nonlinear Finite Element Analysis to assess the anticipated crack widths (see section 2.4).

For combined reinforcement (the “Secondary” case), a methodology for crack width calculation is given by RILEM (2002), based on fracture mechanics principles using the ‘$\sigma-\omega$‘ curve method for strain-softening FRC. Design standards such as the DAfStB Richtlinie Faserbeton (2010), RILEM (2003) and Model Code 2010 contain similar approaches.

2.3.2 Durability of fibre reinforced concrete segments

Precast concrete segments are usually specified as high density concrete with a strength greater than 40 N/mm2. Therefore they are likely to exhibit good durability. FRC segments have been demonstrated to be durable and, when fibres are used with bar reinforcement, the fibres reduce the risk of corrosion of the bars. The longevity of the lining is primarily dependent on its susceptibility to degradation due to physicochemical effects such as sulphate attack, alkali-silica reaction, carbonation and corrosion of the embedded steel reinforcing bars or steel fibres. Macro synthetic fibres themselves are not affected by corrosion.

These degradation mechanisms are directly related to concrete permeability and especially the crack widths. In practice, the design crack widths should be limited depending on the application and the groundwater conditions. The appropriate crack width limits can be taken from the applicable concrete design standard. As noted above, for corrosion protection of steel fibres, a crack width limit of 0.15mm to 0.20mm can be considered as best practice, depending on the aggressivity of the environment (AFTES 2013). A review of research can be found in ACI 544.5R-10 (2010).

For combined reinforcement (the “Secondary” case), the bar reinforcement must be protected. The maximum crack width at SLS has a direct impact on the water tightness of the lining and ingress of chloride ions and other deleterious materials promoting steel corrosion. It should be noted that, with conventional reinforcement, damage at edges and corners often occurs. Due to the minimum cover required for protection against

corrosion, and the shape of the edges, the concrete is unreinforced over a certain thickness, making it vulnerable to damage. Any steel bars exposed to the atmosphere will start to corrode and initiate further spalling, unless this damage is repaired. Because the fibres are uniformly distributed throughout the concrete, FRC segments are much less often damaged in this way and this saves money in repairs as well as reducing the corrosion risks to any bars in the segments.

Considering steel fibres specifically, two different scenarios need to be taken into account when analysing the risk of corrosion of fibres and its consequences:

- Firstly, if the fibres do not cross a crack
- Secondly, if the fibres do cross a crack.

2.3.2.1 The fibres do not cross a crack

As long as the matrix retains its inherent alkalinity and it does not contain large cracks, deterioration of SFRC is not likely to occur. It has been found that, when exposed to conditions conducive to reduced alkalinity, good quality SFRC will only carbonate to a depth of a couple of millimetres over a period of many years (Nemegeer et al 2000). Due to the large surface area to volume ratio, steel fibres are more effectively screened by the lime rich layer than the large diameter bars used in conventional reinforced concrete.

Unlike structural reinforced concrete, SFRC will not support the galvanic corrosion cells. The fibres do not touch each other and so they do not provide a mechanism for propagation of corrosion activity. Furthermore, since the individual fibres are so small, corrosion will not result in any spalling. For example, where the fibres are exposed on the surface of the segments, corrosion will simply result in some minor rust-coloured staining on the surface. If this cosmetic effect is undesirable, galvanised fibre can be used.

2.3.2.2 The fibres do cross a crack

Considering the second scenario of cracked concrete, fibres exposed in a crack will be subjected to corrosion. How long the fibres remain capable of load transfer across the crack and restricting crack widening depends on crack width and depth, type and diameter of fibre used and the aggressivity of the environment. If the crack widths are small enough, corrosion may not occur and the cracks themselves may autogeneously heal. For example, tests conducted on SFRC samples, in a cracked condition, have shown that after 650 cycles of alternating exposure to sea water, there is no loss of bending strength if the crack width is smaller than 0,25mm (Hannant & Edgington (1975) and Mangat & Gurusamy (1987). In another study, no decrease in post-crack strength occurred after 18 months of exposure (in indoor, outdoor, in demineralized water + CO_2 and in salt water + CO_2) if the crack width is smaller than 0.2 mm (Nemegeer et al 2000). Others have found the limiting crack width to prevent corrosion to be lower, around 0.1 mm (Granju & Ullah Balouch 2005 and Kaufmann 2014). Zinc coated fibres show benefits for corrosion resistance (Nemegeer et al 2000). Macro synthetic fibres have been shown to perform well in cracked concrete (Kaufmann 2014).

2.4 NUMERICAL MODELLING OF FRC

2.4.1 General

Numerical modelling using commercially available software (such as Finite Element Analysis) is becoming increasingly popular in tunnel segment design, although the design engineer needs to have thorough training and experience in this field. A general policy used in numerical analysis is that an approximate hand calculation is needed to check the results of the numerical modelling to prevent major errors.

Once the potential for major errors has been eliminated, a numerical model can be useful to generate more accurate estimates of stress actions and deformations than hand calculations, especially for complex geometries.

There are varying levels of sophistication available in numerical modelling. The most basic analysis can be undertaken using linear-elastic models. These will produce good results at a low level of stress (i.e. where plasticity is unlikely to occur). As the level of stress imposed increases, more sophisticated material models are required to simulate the non-linear post-cracking behaviour of fibre reinforced concrete. Inclusion of time-dependent properties such as creep of the concrete or fibres requires a level of sophistication that is presently usually only found in the sphere of research.

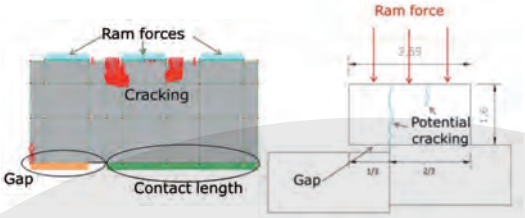

Figure 5: Finite Element modelling of ring joints under jack thrust with differential bedding contact, showing crack development: a) results from the model b) schematic of the out-of-planarity case

2.4.2 Modelling of post crack behaviour

Concrete exhibits quite different behaviour in compression and in tension: on the compression side it exhibits a strain hardening behaviour (up to the ultimate compressive strain) with a relatively high compressive strength compared to tensile strength (~10 times larger), while on the tension side it exhibits a quasi-brittle material behaviour. Furthermore, concrete exhibits creep in response to persistent loading, and because of brittle behaviour does not respond well to dynamic effects.

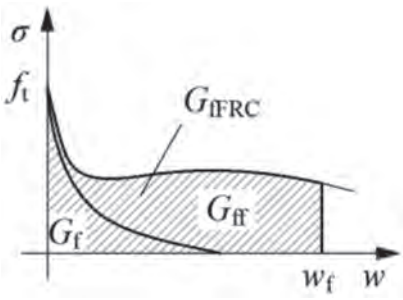

Figure 6: Definitions of fracture energy of concrete (Gf), fibre-reinforced concrete (GfFRC) and the added fracture energy of fibre (Gff)- (Juhasz 2013)

When the induced stresses exceed the tensile strength of the concrete it will crack. There will be a residual stress at the crack surface that depends on the crack width opening distance. This stress is associated with an energy, called fracture energy (Gf). This energy is influenced by the aggregate type (round or crushed), size, and its bond to cement mortar. Fibres increase this fracture energy (Gff), thereby making the concrete a more ductile material (Juhasz 2013).

The most important criterion for the selection of the FRC material model is the ability to model this increased fracture energy (GfFRC) and select a value that is appropriate to the FRC used for a design (see Figure 6). The energy that is associated with the inclusion of fibres in the concrete, Gff, can be measured by different material tests, e.g. 3 point notched beam test with load-CMOD and load-deflection results. The stress-crack width relation required in a numerical modelling can be determined by a back analysis of these test results. Methods of doing this can be found in the literature (e.g. di Prisco et al 2013 and Juhasz 2013).

One possible approach for the use of post-crack fracture energy in numerical modelling is based on the "crack band theory" developed by Bazant (Bazant & Oh, 1984). Generally, this method converts the stress-crack diagram to a stress-strain diagram using the characteristic length (lcs). In the numerical model, the characteristic length is a mesh-dependent variable: its length changes according to the size of the element and the angle of the crack direction. According to the "crack band theory", the appropriate size for the element should be the same as the width of the fracture process zone which is approximately 2.5-3 times the maximum aggregate size. If the numerical modelling software cannot model this phenomenon, substantial errors may result when a post-cracking analysis is attempted.

In summary, the material model used for modelling FRC must include the following:

- A combined failure surface for modelling of peak strength,
- Inclusion of the fracture energy (Gf) parameter for modelling of post-cracking performance,
- The fracture energy can be determined from the back analysis of test results.
- A stress-strain model that incorporates crack band theory to resolve the mesh dependency issue.

2.5 FIRE PROTECTION

Adding monofilament polypropylene (PP) micro-fibres is widely accepted as the best means of passive fire protection for concrete tunnel linings (see Appendix G). These fibres reduce explosive spalling by facilitating the release of steam vapour pressure.

Fibrillated PP fibres provide a limited degree of protection, whilst macro synthetic and steel fibres alone have been found to have little or no influence on the prevention of explosive spalling.

Large scale fire tests are really the only way to determine the correct dosage. This is a costly exercise and an expense that many projects would like to avoid. Section 6.2 of EN 1992-1-2:2004 makes reference to the use of 2 kg/m^3 of monofilament polypropylene micro-fibres to control explosive spalling in high strength concrete. Unfortunately no fibre specification (i.e. diameter or length) is given. This does not preclude the usage of lower or higher dosages, however it does highlight the need for careful consideration and the necessity to carry out fire testing on large concrete samples that completely replicate the materials to be used on an actual project. Where this has been done, dosage rates of, for example, 1.0 kg/m3 and 1.5 kg/m3 have been used in actual tunnel projects. Appendix G contains more information on this subject.

When considering the effect of the fire, EN 1992-1-2:2004 provides methods of calculating the reduction of concrete strength due to high-temperature damage within the concrete and, where applicable, steel reinforcement. The heat from a fire could reduce the strength of steel fibres and destroy macro synthetic fibres. On the other hand, fibres may not be needed for the longterm design loads as the lining is designed to work in compression. Ingason (2006) made recommendations regarding the most appropriate fire temperature curve for a range of fire risks, based on the UPTUN experiments.

3.1 GENERAL

Most of the engineering properties of FRC are primarily related to its concrete matrix properties and thus test methods developed for concrete in both fresh and hardened states can be used in case of FRC. These include the conventional concrete tests at early-age to characterise workability, plastic shrinkage, fibre content and, at later ages, (hardened state) mechanical strength tests such as compression, indirect tensile (splitting) and modulus of elasticity. The pre-construction and production quality control tests required are summarised in AFTES (2013). It should be noted that the tests apply both to the constituent materials of the concrete, and to fresh and hardened concrete.

3.2 TENSILE STRENGTH TESTING

It is the post-crack tensile performance of FRC that marks the significant difference of FRC's behaviour in relation to that of plain concrete. For the design of a segmental FRC tunnel lining, the most important parameter to be quantified is the tensile stress-strain response in its hardened state. As discussed earlier (see sections 1.3.2 and 2.2.4), the tensile strength at first crack is not altered by the fibres and, where needed, it can determined by standard tests.

The determination of the post-crack tensile properties can be achieved with the following types of tensile tests:

- Direct Uniaxial
- Flexural (bending)
- Indirect/Special

The first type of tests is the tensile test of specimens under direct uniaxial load. These are notoriously difficult tests, as it is almost impossible to avoid bending in the medium sized specimens, which are required to minimise the fibre length effect. These tests are used primarily in research as they enable the stress - strain and stress - crack width curves to be derived directly from the testing, without analytical manipulation. One type of tensile test often quoted in technical literature is described in RILEM (2001) and is performed on cylinders 150mm. A number of different variations of uniaxial tensile tests are briefly presented in BRE Digest 451 (2000) publication 'Tension tests for concrete'.

The second type of tests is the most common type. These tests involve beams tested in bending. The beam bending tests include three-point or four-point (or 'third point') tests at beam aspect ratio (length : depth) greater than 3:1. The main advantage of these types of test is that they are well established, relatively easy to carry out and there is an extensive database (on some of them) for comparative studies. The most appropriate and common tests will be presented in the following sections.

The last type of tests refers to a number of specialised tensile tests like the Wedge Split Test (WST, see Lofgren 2005)) and the Double Punch Test (DPT or 'Barcelona' indirect tensile test). The WST provides the stress - crack width curve and is independent of specimen size, although the results are affected by the size (length) of fibres as well as the type of fibre. The DPT is performed on cylinders 150mm high by 150mm in diameter and the derivation of load deflection curve is described in technical literature (Molins et al 2009). However, it should be noted that these tests are specialised and not - at this point in time - validated by any international standards organisation. Therefore, they should not be used as stand-alone tests as an alternative to the standardised beam bending tests or full-scale segment testing.

In summary, testing of FRC's structural performance should be carried out for the following reasons:

- Characterisation of post-crack behaviour in flexure
- Derivation of tensile strength parameters that can be used in design analysis
- Assessment of ductility and toughness indices for comparative studies of different FRC mixes or/and fibre types

The specification should define the types and frequency of this testing as well as the acceptance criteria.

3.3 FLEXURAL STRENGTH TESTING

3.3.1 Introduction

Material classification is an important requirement for verification of the design. When referring to ordinary concrete, designers choose its strength, workability or exposure classes which then have to be provided by the concrete producer. The compressive strength of FRC is not particularly influenced by the presence of fibres (up to a volume fraction of 1% - Lofgren 2005). Hence the normal classification for plain structural concrete can be adopted.

Plain concrete is characterized by a brittle behaviour in tension. The reason for adding fibres to cementitious composites is to improve the tensile behaviour after cracking, in terms of providing a residual tensile strength and ductility. It should be underlined that FRC is a composite material, rather than the fibres being just an addition to a concrete matrix; for this reason a proper mix design is required and mechanical properties should be determined by testing the composite itself.

The ability of FRC to absorb energy beyond the first crack in the matrix is termed "toughness". FRC toughness depends on fibre characteristics (such as material properties: elastic modulus, shape, aspect ratio, tensile yield strength, mechanical anchorage, quantity (usually expressed by the volume percentage %) and orientation as well as on the properties of the cementitious matrix surrounding the fibres.

It should be noted that not all the strengths quoted at deflection values in standard tests may be relevant to the design of tunnel linings. Section 3.3.3 discusses this in more detail. The determination of toughness through testing is also influenced by the test method (load, load rate, load control, stiffness frame), specimen size and instrumentation reliability.

3.3.2 Notched vs unnotched beams

Some commonly used third-point loaded beam test methods such as ASTM C1609/C1609M share similarities in that load is imposed at the third-points, the beam dimensions are usually 150mm x 150mm in cross-section with a span of 450mm, and there is no notch introduced to the specimen prior to testing. The absence of a notch has a major influence on performance compared to notched beam test methods such as EN14561 (Foster et al 2012, Stahli & van Mier 2006). A sawn notch has the effect of increasing both the Modulus of Rupture (alternatively expressed at the Limit of Proportionality in EN14561) and post-cracking residual strengths compared to unnotched specimens. The primary reason why a notch increases apparent performance compared to unnotched specimens and real structures is related to the presence of imperfections in FRC. The notch controls the position of the crack and forces it to occur at a specific location in the specimen regardless of the location of nearby weak points. In contrast, the crack can occur anywhere between the third-points in an ASTM C1609/C1609M beam test and will find the weakest point in an unnotched specimen. X-ray analysis of cracks and fibre density in FRC members (Foster et al, 2012) has shown that an unconstrained propagating crack will divert around the ends of fibres within a stressed FRC member. However, it will be forced through the fibre when constrained by a notch, thus the average post-cracking performance of a third-point loaded unnotched specimen is lower than that of a nominally identical notched specimen. For the type of FRC commonly used in tunnel segments, the magnitude of the difference in apparent post-cracking flexural strength is about 10-20% at equivalent crack widths.

Since in reality FRC tunnel segments do not include notches, the flexural performance of a sample of FRC tested in a notched beam will be higher than the performance obtained in an unnotched beam and the performance of the FRC in the actual structure. In theory, a correction factor should be applied to flexural data generated in notched specimens.

The RILEM and fib MC2010 design methods are based on notched beams because of the perceived benefits of notched samples. These are that the notch will provide a slower cracking process, thereby reducing the risk of a sudden drop in load. Also notch allows the test to be controlled on the basis of the rate of increase of CMOD and the rate of increase of deflection.

With this issue in mind, it is essential that the design method and test method are consistent. This shows that results from different tests cannot be compared directly in some cases. This issue does not affect to data generated using third-point loaded unnotched beams.

3.3.3 Standard Tests for FRC flexural strength testing

3.3.3.1 General

The most common flexural tests for FRC (defined in a standard) are described in the following sections. There are also bending tests, described in various National Standards such as German DIN, Italian UNI, Belgian NBN, etc. These can be adopted for the characterisation, if relevant experience exists within the designer and accredited laboratories can be used for carrying out the bending tests.

A feature of the bending tests compared to full-scale testing of tunnel segments is that the coefficient of variation (COV) on the residual mean values may be quite significant, sometimes up to 25%. This could lead to a signification reduction in design tensile strength after the application of statistical rules for interpretation of test results (see Appendix B). However, with appropriate training for testing staff, this can be mitigated and COVs as low as 10% can be achieved in residual strengths with EN 14651 beams, using 12 specimens per series. The best practice is to make one worker responsible for each type of specimen.

It is important to note that the toughness parameters are sensitive to size effects. This effect is more pronounced in high performance FRC mixes that exhibit deflection hardening behaviour and therefore special care is required, if toughness parameters are to be used in design calculations.

The ASTM C1550 and EN 14488-5 (EFNARC plate) are plate bending tests which are widely using for sprayed concrete lined tunnels but they are not relevant to precast concrete segments.

3.3.3.2 ASTM C1609

This test is performed on a beam (350mm x 100mm x 100mm or 500mm x 150mm x 150mm), without any notch, on a four point loading configuration and it requires a servo-controlled closed-loop machine. This test is used widely in North America and addresses some of the issues with the previous, well-documented ASTM C1018. The ACI 544.FR design method is based on this test (Bakhshi & Nasri 2014). Performance is expressed as a load-deflection curve in which load is taken to be the total load applied across the two third points and the deflection is taken to be the central deflection measured relative to the supports.

The parameters used to summarize performance are defined graphically in Figure 7. Performance is primarily assessed at three points: the first peak in the load-deflection curve, which is used to calculate the Modulus of Rupture; the load at 0.75 mm central deflection, which is used for performance assessment under service conditions (SLS); and the load at 3.0 mm central deflection, which is used for ultimate strength estimates (ULS). These points correspond to the load at first crack and deflections of span/600 and span/150 for 500mm x 150mm x 150mm beams.

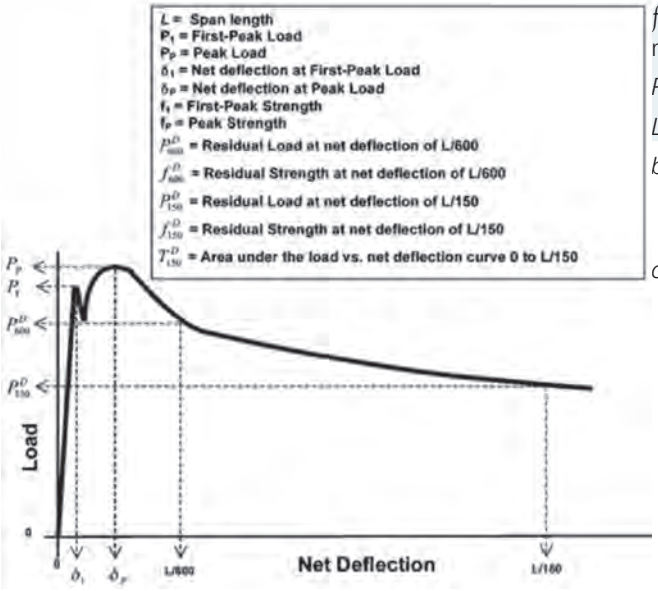

Figure 7: Typical load vs deflection curve for ASTM C1609 for strain softening FRC

While the crack is free to occur anywhere between the third-points, experience has shown that it tends to occur near the centre. For a centrally located crack, the maximum crack width for a deflection of 0.75mm is 1.0mm. For a centrally located crack, the maximum crack width at 3.0mm central deflection is 3.5mm. Steel fibres longer than 50mm are not normally permitted to be used in cast ASTM C1609/ C1609M beams due to fibre alignment problems caused by vibration of the mould walls. If longer steel fibres are used the beam must be sawn from a larger cast specimen or a performance correction factor applied to the results.

Performance parameters are calculated as follows. The first-peak strength is determined using the first-peak load shown in Figure 1, the average specimen dimensions measured at the location of the crack, and the following formula for modulus of rupture:

$$f = \frac{PL}{bd^2}$$

Eq. 1

where:

f = the strength, MPa or N/mm^2

P = the load, N

L = the span length, mm

b = the average width of the specimen at the point of fracture, mm, and

d = the average depth of the specimen at the point of fracture, mm

Determine the peak load as that value of load corresponding to the point on the load-deflection curve that corresponds to the greatest value of load obtained prior to reaching the end-point deflection. Calculate the peak strength using the peak load, the average specimen dimensions determined above, and Eq 1.

Determine the residual load values, P^D_{600} and P^D_{150} for specimen depth of 150mm, corresponding to net deflection values of $L/_{600}$ (0.75mm) and $L/_{150}$ (3.0mm) of the span length. Calculate the residual strengths, f^D_{600} and f^D_{150} using the residual loads, the average specimen dimensions, and Eq 1.

3.3.3.3 EN 14651

This is a test developed specifically to characterise FRC and derive design parameters. EN 14651[4] is the reference standard for the European Union CE label for steel and polymer fibres and has been adopted by a number of fibre manufacturers and designers, primarily in Europe, Asia and Middle East. The great advantage of this test is that it relates the strength to specific CMODs (Crack Mouth Opening Displacement) and the strength indices can be used directly in design for the appropriate Limit State. This test procedure has been adopted by Model Code 2010 and its implementation is relatively straightforward and independent of the type of fibre.

The behaviour in tension of FRC is evaluated in terms of residual flexural tensile strength values, which are determined based on the load-crack opening curve or the load-deflection curve, obtained by applying a centre-point load on a simply supported notched (notch: 25mm length, 5mm wide) beam (550mm x 150mm x 150mm) on a 500mm span. The test results are expressed as the limit of proportionality (LOP) and the residual flexural strength (see Figure 8).

The limit of proportionality, $f_{ct,Lf}$, is calculated as

$$f^f_{ct,L} = \frac{3}{2} \cdot F_L \cdot \frac{l}{bh^2}$$

where F_L is the maximum load between a CMOD of 0 and 0.05mm or a deflection of 0 and 0.08mm. The residual flexural strength, $f_{R,I,}$ needs to be evaluated at four different displacements.

$$f_{R,i} = \frac{3}{2} \cdot F_{R,i} \cdot \frac{l}{bh^2}$$

where $F_{R,i}$ is the residual load at:

- i = 1: CMOD = 0.5mm or deflection 0.47mm
- i = 2: CMOD = 1.5mm or deflection 1.32mm

[4] This is similar to RILEM test, although the RILEM test refers to steel fibre concrete: see TC162TDF 2002, 'Bending Test – Final Recommendation', Materials and Structures, v35, 579-582

- i = 3: CMOD = 2.5mm or deflection 2.17mm
- i = 4: CMOD = 3.5mm or deflection 3.02mm

l = the span between the supports (nominal distance 500mm)

b = the width of the concrete sample (nominal value 150mm)

h = the residual height of the concrete sample (nominal value 125mm)

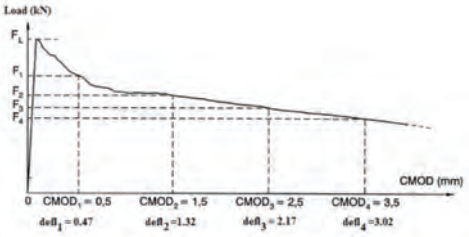

Figure 8: Typical load vs CMOD curve for EN14651

The residual strength indices which are of greater importance, according to fib Model Code, are:

- Value f_{R1} (CMOD = O.5mm) is used for the verification of Service Limit State.
- Value f_{R3} (CMOD = 2.5mm) is used for verification of the Ultimate Limit State.

The FRC stress-strain curve can be derived from the strength indices and a stress block can be deduced assuming either linear-elastic or rigid plastic post crack behaviour. With the previous assumptions, FRC toughness can be classified by using a couple of parameters. The first one is a number representing the f_{R1} class while the second one is a letter representing the ratio f_{R3k}/f_{R1k} The f_{R3k}/f_{R1k} ratio corresponds to different strength classes.

The strength interval for f_{R1k} is defined by two subsequent numbers in the series:

1.0 ; 1.5 ; 2.0 ; 2.5 ; 3.0 ; 4.0 ; 5.0 ; 6.0 ; 7.0 ; 8.0 [N/mm^2]

The f_{R3k}/f_{R1k} ratio can be represented with letters a, b, c, d, e, corresponding to the ranges:

"a" if $0.5 \leq f_{R3k}/f_{R1k} \leq 0.7$

"b" if $0.7 \leq f_{R3k}/f_{R1k} \leq 0.9$

"c" if $0.9 \leq f_{R3k}/f_{R1k} \leq 1.1$

"d" if $1.1 \leq f_{R3k}/f_{R1k} \leq 1.3$

"e" if $1.3 \leq f_{R3k}/f_{R1k}$

A typical example of FRC classification is given below:

FRC 40/50 - 5.0c means

- Compressive cylinder strength, fck = 40 N/mm^2
- Residual flexural strength at CMOD = 0.5mm f_{R1k} = 5.0 N/mm^2
- Residual flexural strength at CMOD = 2.5mm f_{R3k} = 4.5 N/mm^2 up to 5.5N/mm^2

(NB: All strength values are characteristic values after statistical analysis.)

More precise minimum characteristic values can be specified by the designer as a refinement to this classification.

Since brittleness must be avoided in structural behaviour, fibre reinforcement can be used as substitution (even partially) of conventional reinforcement (at ULS), only if both the following relationships are fulfilled (according to fib MC2010):

$f_{R1k}/f_{Lk} > 0.4$ and $f_{R3k}/f_{R1k} > 0.5$

Where f_{Lk} is the characteristic value of the nominal strength, corresponding to the peak load (or the highest load value in the interval 0 – 0.05mm), determined from the EN 14651 beam test.

Typical minimum performance levels, for tunnel segmental linings are:

- Compressive Strength:
  - early-age (demoulding, handling and storing): fck > 12 N/mm^2;
  - at 28 days, fck ≥ 40 N/mm^2
  - at 90 days, fck ≥ 50 N/mm^2
- Bending – residual tensile strength:
  - early-age (demoulding, handling and storing): f_{R1k} > 1.2 N/mm^2;
  - at age equal or greater to 28 days, f_{R1k} > 2.2 N/mm2, f_{R3k} > 1.8 N/mm^2;

These are only indicative performance levels and there may be reasons why a particular project requires higher ones (see Appendix D). More details of the test procedure are included in Appendix H.

An example of a flexural strength vs CMOD curve is shown in Figure 9. The data presented are from a large diameter (7.8m ID) water tunnel in London with a steel fibre reinforced segmental lining. The concrete mix was specified to be resistant against aggressive ground conditions with a ratio $f_{R1k}/f_{Lk} \geq 0.4$ and $f_{Lk} \geq 5.5$N/mm^2. The performance requirements were achieved by using a mix CEM IIB (27% fly ash, w/c < 0.4) C50/60 (at 56 days) with 30kg/m^3 of steel fibres (aspect ratio 80, length 60mm).

The results shown in the graph are based on more than 30 bending tests (according to EN 14651). The mix can be classified as FRC 50/60 - 2.5b.

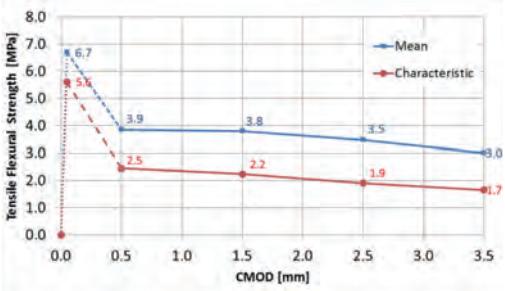

Figure 9: Results from EN14651 tests for steel fibre reinforced concrete segments from a London project

While EN 14651 is used for many international standards such as the Model Code and CE marking, a performance class defined by Model Code can easily be adapted to ASTM 1609 tests (see also the discussion on notched vs unnotched beams in 3.3.2).

- L/150 according to ASTM 1609 equates to about the same deformation as fr4 according to EN 14651
- L/600 according to ASTM equates to about the same deformation between fr1 and f_{r2} according to EN 14651.

3.3.3.4 JSCE SF-4

This used to be a very popular flexural test to derive the toughness of FRC, especially for the design of FRC slabs. It is performed on a beam 450mm x 150mm x 150mm. The total area under the load deflection curve is measured out to a specified deflection (L/150) and a toughness factor is defined. The toughness factor then can be used in the calculation of flexural (bending moment) resistance.

The main disadvantage of this method is that the toughness factor represents an average value of load-bearing capacity over the displacement from zero to L/150. It does not differentiate between pre-peak and post-peak behaviour. That means that FRCs, which exhibit strain softening behaviour, might be significantly overestimated in performance at larger displacements. A further disadvantage of this test is the calculation of the toughness parameter, which is dependent on the specimen size.

3.4 FIBRE SPECIFICATION

This section describes the minimum recommendations proposed for steel and macro synthetic fibres for structural applications.

3.4.1 Steel fibres

Minimum requirements on the properties and for quality controls of steel fibres are defined in ISO 13270, ASTM A820 or EN 14889-1. ISO 13270 has two classes for tolerances, one which is similar to EN 14889 and a more stringent class (Class A), which is often recommended for uses such as precast segments. In EN 14889, a differentiation is made between two systems for the certificate of conformity: system 1 fibres for load-bearing purposes) and system 3 (fibres for other purposes). In countries where EN 14889-1 or ASTM A820 are not applicable, or in countries where no other national standard on steel fibres is issued, the ISO 13270 standard can be used.

The type of fibre and the dosage should be adapted to the compressive strength of the concrete. For cold-drawn wire steel fibres (Group 1 EN 14889-1), a minimum tensile yield strength of steel wire of fy ≥ 800 N/mm^2 is recommended when the concrete class is less than or equal to C40/50 (fck=40N/mm^2). At higher compressive strengths, the tensile strength of the steel fibres has to be higher to maintain ductility. fy ≥ 1500 N/mm^2 is recommended for High Performance Fibre Reinforced Concrete (HPFRC).

In addition, here are some other notes:

- To ensure a maximum network effect (m/m^3) the aspect ratio should be set as follows, l/D ≥ 65. l/D= 80 is recommended;
- The anchor system should be optimised (e.g. using hooks at the end to ensure the fibre is anchored in the concrete matrix);
- The air content with fibres may not differ more than 2% versus plain concrete (ISO 13270);
- Fibre dosage and concrete mixing must take account of the following:
  - Introduction of the fibres using an automatic dosing system which is validated for the selected fibres by the dosing equipment producer
  - Proper mixing of the fibres in the concrete to obtain an uniform distribution
  - Complete elimination of the appearance of fibre balls (detrimental during installation phase)
  - The use of bonded/glued fibres can assist in fulfilling the above requirements.

The normal dosage for steel fibres ranges from 30 to 50 kg/m^3, depending on the performance required for each case.

3.4.2 Macro synthetic fibres

Macro synthetic fibres or any other type of fibres, which comply with EN 14889-2 Class II certification (i.e. diameter > 0.30 mm), can be used as a structural reinforcement. This standard, like those for steel fibres, covers definition, specification and conformity. Fibres can be both mono-filament or fibrillated. The mean tensile yield strength for macro synthetic fibres should be greater than 500 N/mm^2.

The normal dosage for macro synthetic fibres ranges from 8 to 10 kg/m^3, depending on the performance required for each case.

3.4.3 Micro polymer fibres

Micro polymer fibres have no reinforcing effect but they can be successfully used to improve early age concrete shrinkage and fire resistance, even under severe fire conditions (see section 2.5, OVBB guideline and ACI 544.5R-10).

PROPERTY	SYMBOL	DEVIATION OF THE INDIVIDUAL VALUE RELATIVE TO THE DECLARED VALUE	DEVIATION OF THE AVERAGE VALUE RELATIVE TO THE DECLARED VALUE
		Class II	Class II
Length & developed length > 30mm < 30mm	l & l_d	±10% ±10%	±5% ±1.5mm
Equivalent diameter > 0.30mm	d_e	±50%	±5%
Length/diameter	λ	±50%	±10%

Table 1: Tolerance on lenght and diameter for macro synthetic fibres (EN 14489-2)

Increasingly, today's design engineers are required to make conscious efforts to reduce the embodied carbon dioxide (CO_2) contents of the structures they are designing. It is generally recognised that increasing emissions of CO_2 gas, generated from fossil fuelled power supplied to industrial processes and fossil fuelled vehicles, are responsible for global climate change.

Concrete and steel are the most widely used materials in the construction sector and both materials, requiring high consumption of power in their manufacture, are responsible for very large CO_2 emissions into the earth's atmosphere.

Concrete is the second most widely consumed commodity on the planet (after water) and the manufacture of Portland Cement is the largest chemical processing industry in the world today. Table 2 shows figures for the embedded CO_2 contents that were published by the Concrete Centre of the United Kingdom in its "Fact Sheet 18 [P1] – Embodied CO_2 of UK cement, additions and cementitious material" (2007).

CEMENT AND ADDITIONS	EMBODIED CO_2 (KG CO_2/TONNE)
Ordinary Portland Cement	930
Ground Granulated Blast Furnace Slag (GGBFS)	52
Pulverised Fuel Ash (PFA or Fly Ash)	4
Limestone	32
Minor additional constituents	32

Table 2: Embodied carbon dioxide for concrete constituents

While it is possible to significantly reduce the embodied CO_2 of a concrete mixture for segment production by replacing a portion of its cement content with GGBFS or PFA, there is little or no difference between the cementitious blends and contents required for the production of fibre reinforced or conventionally reinforced concrete segments for tunnel linings. In fact, durability assessments arising from ground and groundwater conditions (e.g., chlorides and sulphates) may well necessitate a high level of replacement materials for both types of segment.

The use of fibre reinforcement in the design and manufacture of precast concrete segments for tunnel linings can, however, offer significant reductions in the embodied CO_2 of these elements.

Table 3 shows the embodied CO_2 contents of conventional reinforcement, steel wire fibres and synthetic fibres. There are differences in dosage rates of these materials, depending on the project requirements, and differences in power supply sources for their manufacture (coal, gas, oil and nuclear power stations). Additionally, the type of furnace (basic oxygen or electric arc) has a significant effect on the embodied CO_2 of steel fibres. This is reflected in the ranges shown in Table 3.

It may be seen from Table 3 that the use of fibre reinforcement as an alternative to conventional reinforcement cages in the design and manufacture of precast concrete segments for tunnel linings will dramatically reduce their embodied CO_2 footprint.

Designers should remember that the design of a tunnel lining segment is primarily dependent upon the load conditions applied to it (ground loads, handling, transportation and erection, as well as live loads). The table above is not based on a particular performance and is indicative only. The benefits of reduced embodied CO_2 can only be realised when the selected reinforcement methodology is able to meet all the design conditions.

MATERIAL	EMBODIED CO_2 (KG CO_2/TONNE)	DOSAGE USED IN SEGMENT PRODUCTION (KG/m³)	EMBODIED CO_2 PER m³ OF CONCRETE (KG/m³)
Conventional steel reinforcement	1,932 (1)	60 to 160	116 to 309
Steel wire fibres	2,425 (2)	25 to 40	61 to 97
Polypropylene synthetic fibres	260 (3)	8 to 10	2 to 2.6

Table 3: Embodied carbon dioxide for difference forms of reinforcement

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(3) Correspondence from synthetic manufacturers, 2014

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(see also sections 1.1)

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The section reproduces parts of fib's Model Code 2010, defining key design parameters. The structural design of FRC elements is based on the post-cracking residual strength provided by the fibres. For structural use, a minimum mechanical performance of FRC must be guaranteed. Fibres can partially or totally replace conventional reinforcement. Other cases, like early age crack- control or fire resistance, are considered non-structural uses of FRC. Fibres can be used to improve the behaviour at SLS since they can reduce crack spacing and crack width, thereby improving durability. While the mechanical properties of a cementitious matrix are modified when fibres are added, the elastic properties and compressive strength are not significantly affected by fibres, unless a high percentage of fibres is used.

There are two key chapters in fib MC2010:

- Materials: Fibres / Fibre Reinforced Concrete
- Design: Verification of safety and serviceability of FRC structures

STRENGTH AND DUCTILITY CLASSIFICATION FOR FIBRE REINFORCED CONCRETE

f_{R1k} is the reference value for the classification and it can be given the following values: 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0 N/mm^2

Parameter f_{R3k}/f_{R1k}

a. if $0.5 \leq f_{R3k}/f_{R1k} \leq 0.7$

b. if $0.7 \leq f_{R3k}/f_{R1k} \leq 0.9$

c. if $0.9 \leq f_{R3k}/f_{R1k} \leq 1.1$

d. if $1.1 \leq f_{R3k}/f_{R1k} \leq 1.3$

e. if $f_{R3k}/f_{R1k} \geq 1.3$

The figure 10 opposite shows a concrete which could be classified as C 35/45 4.0c.

Fibre reinforcement can substitute (fully or partially) conventional reinforcement at ultimate limit state, if the following relationships are fulfilled:

$f_{R1k}/f_{Lk} > 0.4$

$f_{R3k}/f_{R1k} > 0.5$

Fib MC2010 recognises that some fibres are made from materials whose properties can change significantly over time or due to changes in temperatures. In these cases, special consideration is required to see how or if this design approach can be used.

Constitutive law for ULS

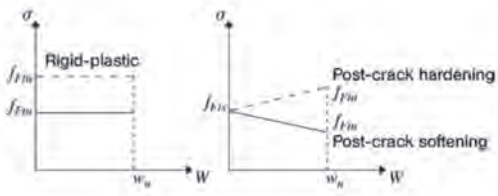

Figure 11: Simplified post-cracking constitutive laws: stress-crack opening (continuous and dashed lines refer to softening and hardening post-cracking behaviour respectively)

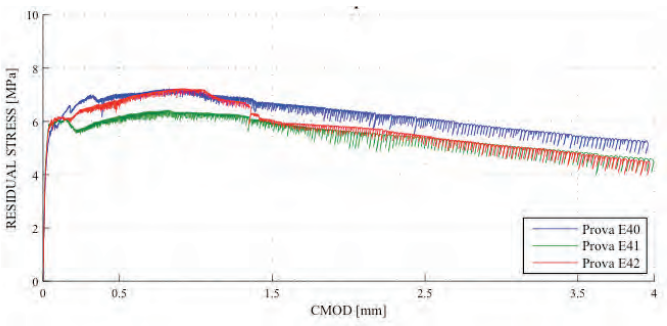

Figure 10: Typical test data for a C35/45 4.0c concrete

Partial safety factor

Design values for the post-cracking strength parameter at ULS can be determined as shown in Figure 11.

The recommended values for partial safety factors are given in Table 5 of Appendix B.

Constitutive law for SLS

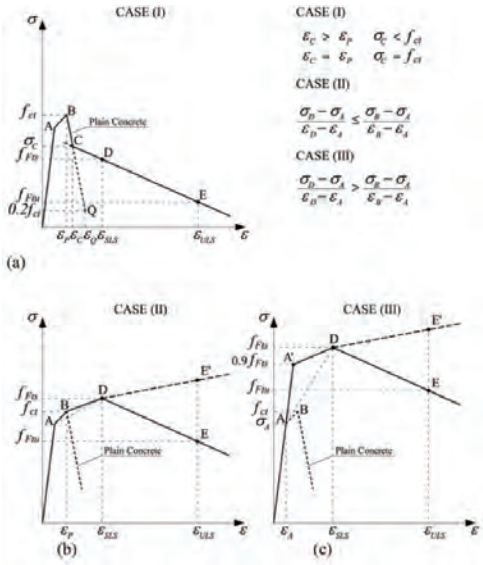

Figure 12: Stress-strain relations at SLS for softening (a) and softening or hardening (b & c) behaviour of FRC.

Structural design must satisfy requirements for resistance and serviceability for the expected life of the FRC elements. The ductility requirement in bending can be satisfied by minimum reinforcement (see 7.7.2 and 7.13 of fib MC2010).

In all FRC structures without minimum conventional reinforcement, one of the following conditions shall be satisfied:

Where δ u is the ultimate displacement, δ peak is the displacement at maximum load and δ sls is the displacement at maximum service load, computed by performing a linear elastic analysis, assuming uncracked concrete and the initial elastic modulus.

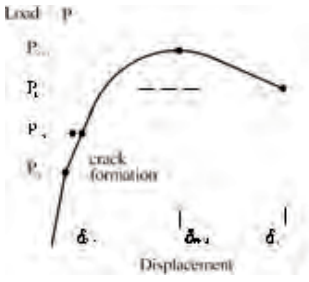

Figure 13: Typical load (P) – displacement (δ) curve for a FRC structure

LIMIT STATE DESIGN

The modern structural concrete design codes have adopted the concept of Limit State, which is a condition where the structure ceases to fulfill the intended function. Limit state design accounts for the factors that influence the resistance (strength of the structure) and the forces (loads) by using a probabilistic basis to reflect the uncertainty in the design calculations. Limit State design requires the use of representative values for the strength of the material, which in the context of limit state standards is defined as 'Characteristic' strength.

CHARACTERISTIC STRENGTH

According to Eurocode 0 (EN 1990:2002) cl. 1.5.4.1 the characteristic value (Xk or Rk) is defined as: "...value of a material or product property having a prescribed probability of not being attained in a hypothetical unlimited test series. This value generally corresponds to a specified fractile of the assumed statistical distribution of the particular property of the material or product. A nominal value is used as the characteristic value in some circumstances."

The strength of the FRC is a fundamental property that determines the structural reliability, and therefore representative values should be used in design. In design calculations the flexural tensile strength of the FRC is represented by the characteristic value, which corresponds to the prescribed probability not being infringed. The characteristic value of FRC flexural tensile strength shall be determined by the following equation:

$$f_{fctk} = f_{fctm} \cdot * (1 - k_n \cdot * V_x)$$

f_{fctk}: characteristic FRC flexural tensile strength

f_{fctm}: mean FRC flexural tensile strength

V_x: coefficient of variation, equal to the ratio of standard deviation of ffct to the mean, ffctm

k_n: is a statistical coefficient depending on whether the variability is known in advance

NUMBER OF SAMPLES	KN,MEAN FOR 95% (KN =1.64·N$^{-0.5}$)	KN,MEAN FOR 5% (KN =1.64·N$^{-0.5}$)	KN,MEAN FOR 5%
	(Vx: known)	(Vx: known)	(Vx: unknown)
3	0.95	1.89	3.37
6	0.67	1.77	2.18
12	0.47	1.71	1.89
>30	0.00	1.64	1.64

Table 4: Coefficient kn for known VX (assuming Normal distribution)

This definition requires that there is a probability of 95% that the mean strength governing the occurrence of limit state in the structure is larger than the characteristic value. For FRC a large database of beam test results exist with a significant number of published data, the coefficient kn is given in Table 4. It is recommended that the minimum number of tests required for verification of the variation coefficient for a new project is 12.

For SLS/ULS verification[5], the characteristic strength shall be based on 5% lower fractile estimate.

Maximum allowable variation coefficient for Beam (EN 14651) tests results is recommended to be 15% and 25%, for fLOP and for f_{R1} to f_{R4} indices, respectively. It is well documented that the bending tests may exhibit significant scatter in the residual values. A higher fractile (say 10%) for deriving the characteristic residual values can be accepted, but this needs to be assessed on each project separately, considering also that a tunnel lining is an indeterminate structure with a significant degree of redundancy allowing stress redistribution. Therefore, the size of the test specimen and the size and type of the structure to be designed should be considered by checking the total area (or volume) where cracking is expected, in relation to the size of the lining . This issue can be partially addressed by carrying out large-scale structural (bending) tests of segments or beams at full thickness.

For SLS verification referring to crack width calculations, the characteristic strength can be based on the 95% reliable estimate of the mean value.

Example:

12 no. beams were cast and tested at 28 days according to EN 14651 and the matrix of the SFRC mix has been classified as C40/50 strength class. At the LOP the strength index f_{LOPm} is 6.0 N/mm^2 with a standard deviation of 0.6 N/mm^2. This means that V_X = 10%. At CMOD = 0.5mm the strength index f_{R1m} is 5.0 N/mm^2 with a standard deviation of 1.25 N/mm^2. This means that Vx = 25%. Hence, based on 12 no. beams, the characteristic strengths are:

At SLS (crack control):

f_{LOPk} =6.0 N/mm^2·*(1 - 0.47 *·0.10) = 5.7 N/mm^2

and

f_{R1k} = 5.0 N/mm^2·*(1 - 0.47 *·0.25) = 4.4 N/mm^2

At SLS/ULS:

f_{LOPk} =6.0 N/mm^2·*(1 – 1.71 *·0.10) = 5.0 N/mm^2

and

f_{R1k} = 5.0 N/mm^2·*(1 - 1.71·* 0.25) = 2.9 N/mm$_2$

[5] These include the typical SLS (deflection, durability) and ULS (stability, fatigue, fire and earthquake)

[6] It has been shown that the scatter of test results reduces with increasing the size of member, see Lambrechts, A. N., 2004. 'The variation of steel fibre concrete characteristics - Study on toughness results 2002-2003', Int- Workshop on advances in FRC, Bergamo, Italy, 135-148

If a significant number of test results are available (>30) from previous projects on the same FRC mix (tested on same facilities), assuming a coefficient of variation, V_X, as 'known'[7], then the characteristic strengths are:

At SLS (crack control):

f_{LOPk} =6.0 N/mm^2·*(1 – 0.00 *·0.10) = 6.0 N/mm^2

and

f_{R1k} = 5.0 N/mm^2·*(1 - 0.00·* 0.25) = 5.0 N/mm^2

At SLS/ULS:

f_{LOPk} =6.0 N/mm^2·*(1 – 1.64 *·0.10) = 5.0 N/mm^2

and

f_{R1k} = 5.0 N/mm^2·*(1 - 1.64·* 0.25) = 2.95 N/mm^2

DESIGN (TENSILE) STRENGTH

The design strength to be used for verifying ULS and SLS can be determined by factoring the characteristic strength to account for uncertainties in structural resistance model, material properties and imposed actions. The characteristic strength should be derived from bending tests such as for example, EN14651.

An alternative approach is to directly assess the representative value for the Limit State examined (when bending test results on a specific FRC Mix are available) and deduce the strength from large-scale testing (i.e. crushing tests on PC segments) at full lining thickness. The partial safety factor in that case can be adopted as γM =1.0.

The designer should select the appropriate approach depending on the available information and the project.

The first approach is the most common way to derive the design tensile strength f_{fctd}, and requires utilizing partial factors of safety on the characteristic flexural strength, f_{fctk}, according to:

$$f_{fctd} = f_{fctk} / \gamma_M$$

where, γM is for material factor to account for model, geometrical and mix uncertainties. FRC for tunnel lining applications remains primarily a cementitious composite and it is recognised that the same material factors as per conventional reinforced concrete should be applied to achieve similar reliability levels. Table 5 summarises the typical values for material factor of safety.

Design Situations or Load Cases (ULS)	Material Factor γ_M
Persistent (Permanent)	1.5
Persistent (high quality control in production & less than 10% variation coefficient on strength tests)	1.4
Transient (Temporary)	1.3
Accidental	1.2
Seismic	1.2

Table 5: Partial Factors of Safety accounting for material strength and structural uncertainties.

For serviceability limit states (SLS) the recommended value is 1.0.

[7] This is a procedure compliant to Eurocode 0 (EN 1990:2002), see Annex D7.1 (5) and appropriate for concrete produced in a controlled environment.

A case study from the Monte Lirio water tunnel, in Panama, is presented here in order to show how full scale tests can be used to assist the design procedure. More information can be found in Meda et al 2013.

Two different kinds of full scale tests have been performed: a) bending tests aiming to evaluate the bearing capacity under flexure, under temporary cases (i.e. demoulding, storage and moving phase) and in field due to the asymmetrical soil pressure and b) point load tests to examine the action of TBM rams on the segment during the excavation process.

The Monte Lirio tunnel is 7878 m long with an external diameter of 3.7 m. The thickness of the precast segments, is 250 mm, so the internal diameter of the tunnel is 3.2 m. Four different segments are used for the lining ring. The S1 precast segment analysed is here, see Figure 14.

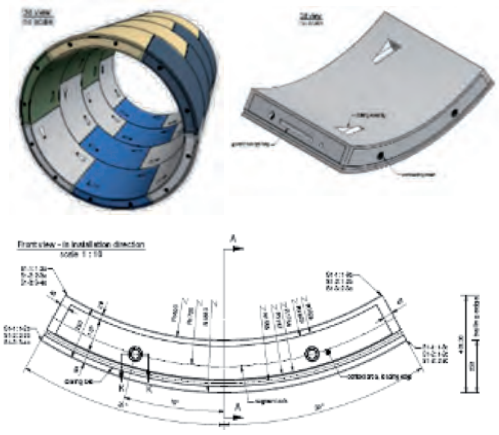

Figure 14: Segment geometry

BENDING TESTS

The segments were placed on a roller support with a span of 1200 mm and the load, applied at midspan, was transversally distributed via a steel beam as shown in Figure 15. During the test, the following parameters were continuously registered:

- The load F;
- The midspan displacement measured by means of four potentiometer wire transducers placed along the transverse line;
- The crack opening at midspan, measured by means of two LVDTs.

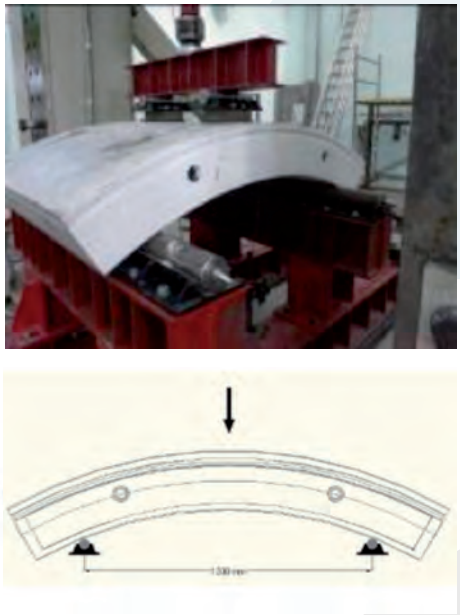

Figure 15: Bending test set-up

The results, expressed in terms of graphs of force vs mean displacement, from the two tests, named BK1 and BK2, are shown in Figure 16. Figure 17 shows the typical cracking pattern.

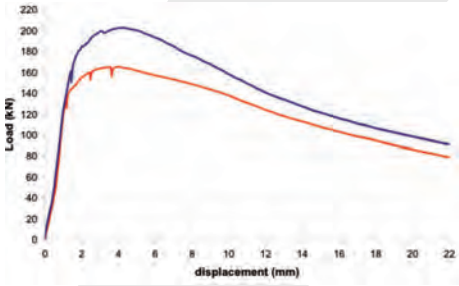

Figure 16: Bending tests: load-displacement diagrams

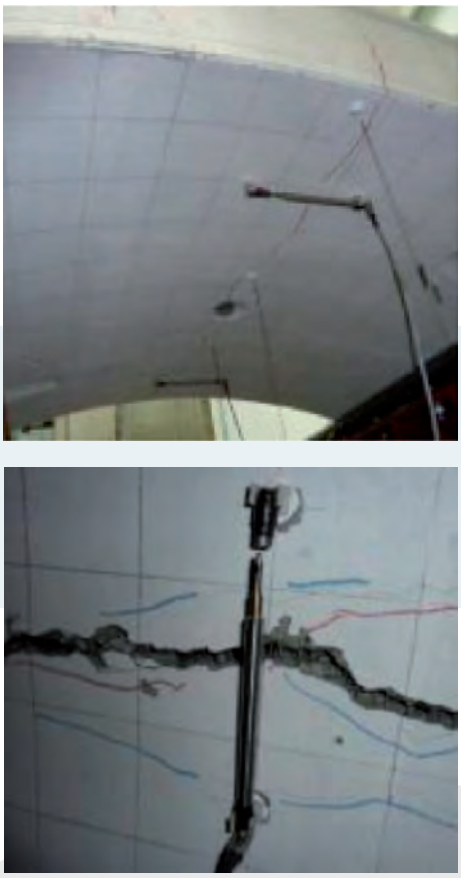

Figure 17: Crack pattern at:
a) first crack
b) failure condition

POINT LOAD TEST

A point load test was performed by applying two point loads at the segment, with the same steel plates used by the TBM machine, see Figure 18. A uniform support was provided by placing the segment on a suitably stiff beam. Two 2000kN jacks were used for every steel plate.

The load was continuously measured by pressure transducers. Four wire transducers (two located at the intrados and two at the extrados) measured the displacement of the shoes, while one LVDT transducer was attached between the shoes, in order to measure the crack openings.

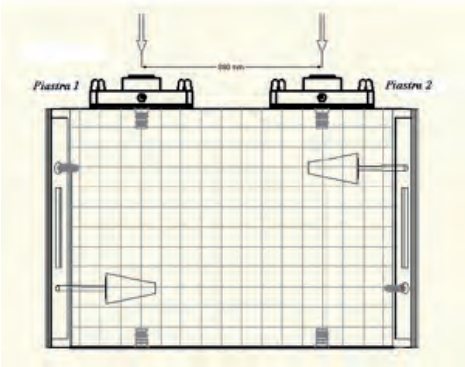

Figure 18: Point load test set-up

Four load steps were imposed:

I- a load step up to 100 kN for the system arrangement;

II- a load step up to 785 kN for each shoe, representing the design limit value;

III- a load step up to 1100 kN for each shoe, representing the maximum load of the TBM jacks;

IV- a load step up to 2000 kN for each shoe (maximum load of the laboratory jacks).

The first cracks appeared for a load of 1650 kN (for each shoes) between the two shoes, as shown in Figure 19 (a). The crack pattern at the maximum load, equal to 2000 kN (for each shoe) is depicted in Figure 19 (b), and shows a small increase in the length of the two cracks between the load shoes.

It should be noted that the first crack occurred at a load level (1650 kN), which is more than twice as high as the design value (785 kN) and higher than the maximum load of the TBM (1100 kN). Furthermore, it can be noted that fibre reinforcement was able to control crack development, with a limited increase of the crack length when the load was increased from 1650 kN to 2000kN.

Figure 19: Point load test:
a) first cracks (1650 kN)
b) crack pattern at load level of 2000 kN

1. PROJECT REFERENCE OENZBERG TUNNEL

1.1 Project Details

History & brief description:
The most complicated part of the tunnel section was the construction of the crossing of the new double track Zurich – Bern line and the existing single track Solothurn line. The first part of the tunnel was driven in hard rock, but a section of 340 m had to be built in a weak soil (Moraine) under the groundwater level. This was called the HYDRO part. In this latter part the TBM progressed at a speed of 170 m per month. The concrete segments are steel fibre reinforced in 2 sections of the tunnel:

- The crossing zone of the Oenzberg tunnel and the Wolfacher South tunnel
- The HYDRO part

Year: 2002

Client and Location:
Swiss Federal Railways, Switzerland

Type of tunnel:
Double track railway tunnel

Ground conditions:
Hard rock/weak soil/Molasse/Moraine

Alignment length:
3160m; Depth=shallow

TBM type: Mixshield TBM; Maximum thrust: the segments were tested and could withstand more than a total ram load of 300MN per ring; the load at collapse equated to about 750 MN per ring.

1.2 Design Approach Adopted

Design method & standard used:
Fibres chosen based on SN 562 162/2 & JSCE-SF4 tests

Specified strengths of concrete – compressive and residual tensile:
C35/45; f_{ctf}~0.7 N/mm^2

Inner and outer diameter:
ID 11,44 m, OD 12,04m

Ring segmentation: 7

Dimensions of segments:
Segment Width: 1,7m, Thickness: 0.300m,

Type of segment reinforcement:
Dramix 3D 65/60BG/ & bar reinforcement

Quantity of reinforcement per m^3 of concrete :
30 kg/m^3 of fibres with 65kg/m^3 of bars for crossing zone & 60 kg/m^3 of fibres only for HYDRO section

1.3 Project key points

- The options were investigated: bar reinforcement (90 kg/m^3); 60 kg/m^3 steel fibres & hybrid bar – fibre solution (30 kg/m^3 of fibres with 65kg/m^3 of bars).
- The research and testing was carried out by Prof. Dr. R. Suter at the Ecole d'Ingénieurs et d'architectes in Fribourg in order to investigate the capacity of SFRC segments.
- Tests were made on the circumferential (longitudinal) joint between adjacent rings with a linear load applied over the entire length of the elements and at the contact between the two rounded joint surfaces at the radial joint.
- All three options gave similar results in full scale compression tests on radial and circumferential joints and in the bending test (see picture below)
- The use of steel fibres instead of standard reinforcement has reduced the number of segments to be repaired as the steel fibres distribute and resist much better the highly concentrated jack loads.
- This extensive test programme resulted in the use of 60 kg/m^3 fibres as the sole reinforcement for the HYDRO part segments of the Oenzberg tunnel

1.4 Picture Reference

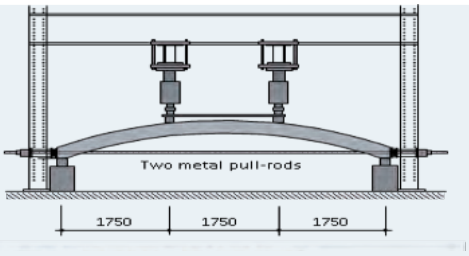

Figure 20: Flexural tests on tunnel segment

2. PROJECT REFERENCE ABU DHABI STEP SEWER PROJECT

2.1 Project Details

History & brief description:
The Abu Dhabi Strategic Tunnel Enhancement Program (STEP) is to provide a major improvement in the capacity of Abu Dhabi's waste water system.

Year: 2009-2015

Client and Location:
Abu Dhabi Sewerage Services Company (ADSSC); Abu Dhabi

Type of tunnel: Sewer

Ground conditions:
Dolomitic Mudstone/Claystone and Gypsum

Alignment length:
45 km. Depth=36-62m

TBM type: EPBM; operating thrust of 3167 kN/shoe with 16 shoes; maximum thrust 54288 kN

2.2 Design Approach Adopted

Design method & standard used:
DBV; Eurocode 2; EN 14889-1

Specified strengths of concrete – compressive and residual tensile:
C50/60 FL 1.6/1.0

Inner and outer diameter:
ID 2.75m, OD 3.31m

Ring segmentation:
5 segments +1 key

Dimensions of segments:
Segment Width: 1,4m ; thickness: 0.280m

Type of segment reinforcement:
Bars at radial joints: stirrups T10@100 with 4 no. T10 bars for bending

Quantity of steel fibres per m³ of concrete:
40 kg/m³ Dramix fibres

2.3 Project key points

- The chloride levels in the ground and ground water are considerably higher than in sea water. In order to deal with this severe environment, steel fibre reinforcement was chosen.
- Uncracked steel fibre reinforced concrete is known to be very durable. Corrosion is limited to fibres exposed on the surface and this does not damage the concrete.
- Chlorides were a particular concern with concentrations up to 9%. Some studies have shown that the durability of steel fibre reinforced concrete under chloride exposure is superior to that of steel bar reinforced concrete.
- An inner lining with a HDPE membrane was installed inside to protect the segmental lining from microbiologically induced corrosion due to aerobic bacterial activity (producing sulphuric acid).
- The design life 80 years.

2.4 Picture Reference

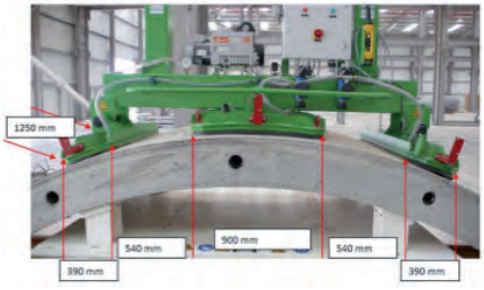

Figure 21: Vacuum lifting system

Figure 22: Stacked segments

3. PROJECT REFERENCE BRISBANE AIRPORT LINK SEGMENTAL LINING, AUSTRALIA

3.1 Project Details

History & brief description:
The Airport Link project involves 15 km of tunnelling including the road (5.7 km of twin tunnels), busway tunnels and connecting ramps, as well as 25 bridges and results in over 7 km of new roads.

More than 300 cubic metres of concrete were used per day to produce more than 100 segments in a 24-hour shift. Over a 12-month period, 65,000 cubic metres of concrete were required to produce the 21,000 segments.

Year: 2008-2013

Client and Location:
BrisConnections, Australia

Type of tunnel: Road

Ground conditions:
Mixed conditions; rock and soft ground; weathered sedimentary silts and sandstones

Alignment length:
5.7km. Depth = 60m

TBM type:
EPBM; Operating thrust: 60000kN (19 shoes), Maximum theoretic al thrust 89000kN

3.2 Design Approach Adopted

Design method & standard used: -

Specified strengths of concrete – compressive and residual tensile:
C55/67; LOP = 7.0MPa, Flexural strength= 3,4MPa

Inner and outer diameter:
ID 11.34m, OD 12.14m

Ring segmentation:
Varies, max=9+1

Dimensions of segments:
Thickness: 0.4m, ring length: 2m

Type of segment reinforcement:
Steel fibres / Micro synthetic fibre

Quantity of reinforcement per m3 of concrete:
35kg/m^3 Dramix RC80/60BN & 1,5kg/m^3 Duomix M6 Fire

3.3 Project key points

Considerable refinement was required to arrive at a mix that provided the required demoulding strength while still providing sufficient ductility to meet the flexural strength requirement. This was achieved by:

- Increasing the proportion of fly ash in the binder from 20% to nearly 29%
- Controlling the compressive strength to keep the average closer to 65MPa
- Adjusting the proportions of sand and aggregate in the mix to improve pull-out performance of the fibres

Key learning points:

- The use of SFRC, rather than conventional bar reinforced concrete, is known to provide significant benefits for long term durability and maintenance of segmental tunnel linings
- Programme/cost savings from reduction/elimination of conventional bar reinforcement by using Steel Fibre Reinforced Concrete (SFRC) segments

4. PROJECT REFERENCE HOBSON BAY SEWER TUNNEL, NEW ZEALAND

4.1 Project Details

History & brief description:
Undersea sewer tunnel to replace a 90 year old viaduct to meet the demands of population growth and to virtually eliminate overflows into the Hobson Bay. First EPB machine driven tunnel in New Zealand.

Year: 2008 – 2009

Client and Location:
Watercare, Auckland, New Zealand

Type of tunnel: Sewer

Ground conditions:
East Coast Bay Formation (weak interbedded sandstone and mudstone), Old Alluvium.

Alignment length:
3.0km; Depth=23m

TBM type:
EPBM. Operating thrust: 12000kN (12 shoes), Maximum theoretic al thrust 21000kN.

4.2 Design Approach Adopted

Design method & standard used:
RILEM TC162-TDF, DBV 2001, EN 14651

Specified strengths of concrete – compressive and residual tensile:
C50/60; LOP = 7.0MPa, fR1 = 3.5MPa, fR4 = 3.0MPa

Inner and outer diameter:
ID 3.70m, OD 4.20m

Ring segmentation:
4 rhombic + 2 trapezoidal (60°, universal rings)

Dimensions of segments:
Developed centreline length: 2.068m, thickness: 0.250m, aspect ratio: 8.3, ring length: 1.20m

Type of segment reinforcement:
Steel fibres

Fibre type: Length, aspect ratio, tensile strength:
L = 50mm, L/D = 67, fy = 1230MPa

Quantity of reinforcement per m³ of concrete:
40 or 45kg Wirand FF3 fibres (depending on ground conditions)

Curing:
4 hours steam curing

Handling and erection:
Vacuum lifter, mechanical erector

Way of stacking (horizontal, vertical):
Vertical

Durability considerations:
35mm sacrificial layer considered in the structural design.

4.3 Project Benefits

- 100% higher productivity by replacing the rebar cages and using a carousel line
- Better segment quality and robustness
- Reduced rejection and repair rates (<0.1%)
- Inherent cost savings (10% of project costs)
- Higher durability of the segments
- Reduced quantity of steel reinforcement and embodied carbon footprint
- The shallower tunnel alignment of the TBM tunnel reduced pumping requirements compared to the initial design, reducing carbon emission by 69,000t of CO2 over the lifetime of 100 years.

4.4 Picture Reference

Figure 23: Project Hobson – Segment production

Figure 24: Project Hobson – Nearing completion

5. PROJECT REFERENCE CONTRACT C933, MRT DOWNTOWN LINE 3, SINGAPORE

5.1 Project Details

History & brief description:
Downtown Line Stage 3 is an underground Mass Rapid Transit (MRT) system consisting of 16 stations along a 21 kilometre route. Package C933 has been selected to trial new lining technologies. This is the first metro tunnel in Asia built using steel fibres as primary reinforcement and micro-synthetic fibres for fire resistance of the segmental lining.

Year: 2012– 2014

Client and Location:
Land Transport Authority (LTA), Singapore.

Type of tunnel:
Mass Rapid Transit (Metro)

Ground conditions:
Old Alluvium and some areas in the Kallang formation

Alignment length:
2.35km; Depth=27,7m - 39,5m.

TBM type:
EPBM. Operating thrust 16000kN-22000kN. Maximum theoretical thrust 35000kN.

5.2 Design Approach Adopted

Design method & standard used:
RILEM TC162-TDF, EN 14651

Specified strengths of concrete – compressive and residual tensile:
C50/60; fLk = 4.2 N/mm^2, fR1k = 2.8 N/mm^2, fR4k = 1.4 N/mm^2

Inner and outer diameter:
ID 5.80m, OD 6.35m

Ring segmentation:
7 segments + 1 key (49.091° and 16.364°)

Dimensions of segments:
Developed centreline length: 2.603m, thickness: 0.275m, aspect ratio: 9.5, ring length:1.40m

Type of segment reinforcement:
Steel fibres and micro-synthetic fibres

Fibre type: Length, aspect ratio, tensile strength:
Steel: L = 50mm, L/D = 67, fy = 1230 N/mm^2
Micro-PP: L = 6mm, D = 18 micron

Quantity of reinforcement per m^3 of concrete:
40kg/m^3 Wirand FF3 steel fibres, 1.0kg/m^3 FibroMac micro-synthetic fibres

Curing:
10 hours heat curing

Handling and erection:
Vacuum lifters

Way of stacking (horizontal, vertical):
Vertical

Durability considerations:
70% GGBS to be used within the binders to minimise chloride penetration

5.3 Project Benefits

5.3.1 Key benefits to the client that led to the decision using FRC

- Higher durability
- Higher productivity
- Reduced work force by using a carousel line

5.3.2 Further benefits

- Better segment quality and robustness
- Reduced rejection and repair rates
- Inherent cost savings
- Reduced quantity of steel reinforcement and embodied carbon footprint
- The successful progress of this pilot project gave the confidence for further projects in South-East Asia.

5.4 Picture Reference

Figure 25: DTL3 C933 – East TBM launch shaft at Bendemeer station

Figure 26: DTL3 C933 – View into tunnel (TBM #3)

6. PROJECT REFERENCE SAN FRANCISCO BAY UTILITY TUNNEL, USA

6.1 Project Details

History & brief description:
San Francisco Bay Utility Tunnel is the first TBM tunnel to be excavated under San Francisco Bay. A key structure in terms of challenges regarding both environmental protection and seismic reliability for the delivery of potable water to millions of customers. Hence there were high performance requirements for the tunnel segments in terms of crack opening and post-crack residual strength.

Year: 2011– 2012

Client and Location:
San Francisco Public Utilities Commission; San Francisco, USA

Type of tunnel:
Utility Tunnel

Ground condition:
Sandy and silty clays under high groundwater pressures (3,45bar max hydrostatic pressure)

Alignment length:
8 km; depth=23 - 34m

TBM type:
Operating thrust ~60%*20000=12000kN

Maximum theoretical thrust 20000kN

6.2 Design Approach Adopted

Design method & standard used:
ACI 318-05, ASTM C1609

Specified strengths of concrete – compressive and residual tensile:
42 N/mm^2; Flexural tensile strength=0,37*3,1N/mm^2

Inner and outer diameter:
I.D. 3.9m, O.D. 4.42m

Ring segmentation:
5 segments + 1 key

Dimensions of segments:
Developed centreline length: 2.275m, thickness: 0.25m, aspect ratio: 9.1, ring length:1.5m

Type of segment reinforcement:
Steel fibres only

Fibre type: Length, aspect ratio, tensile strength:
Steel: L = 50mm, D = 0.75mm, L/D = 67, fy = 1230 N/mm^2

Quantity of reinforcement per m^3 of concrete:
40kg Wirand FF3 steel fibre

Curing:
7 hours heat curing

Way of stacking (horizontal, vertical):
Horizontal

Durability considerations:

6.3 Project Benefits

The San Francisco Bay Utility Tunnel will ensure the provision of reliable, clean and affordable water to the 2.4 million customers in San Francisco, Alameda, Santa Clara and San Mateo. It is designed to replace the existing pipelines which cross the bay on a system of trestles. By tunnelling, the solution enhances the aesthetics of the bay, as well as reducing the vulnerability of the pipeline to earthquakes. A double shell lining concept was used.

6.4 Picture Reference

Figure 27: Project San Francisco Bay Segment production

Figure 28: Segments at the production yard

7. PROJECT REFERENCE SOUTHALL TO HAREFIELD GAS PIPELINE, UK

7.1 Project Details

History & brief description:
The Harefield to Southall Gas Pipeline is a 27 bar, 18.5km gas pipeline in Middlesex, UK, commissioned by National Grid to meet rising gas demands in West London. This included a 900m long TBM driven tunnel with a precast fibre reinforced segmental lining.

Year: 2009

Client and Location:
National Grid; Southall to Harefield, UK

Type of tunnel:
Gas transfer tunnel

Ground conditions:
London clay (a stiff overconsolidated clay)

Alignment length:
976m; Depth approximately 21m

Thrust of TBM:
Maximum theoretical thrust: 6800kN.
Operating thrust: 2500kN

7.2 Design Approach Adopted

Design method & standard used:

Specified strengths of concrete – compressive and residual tensile:
C45/55; flexural strength=5,0 N/mm^2; Residual post-crack=2,4 N/mm^2

Inner and outer diameter:
ID 2.59m, OD 2.95m

Ring segmentation:
7 segments +1 key

Dimensions of segments :
Thickness: 0.180m, ring length: 1.0m

Type of segment reinforcement:
macrosynthetic fibres

Fibre type: Length, aspect ratio, tensile strength: -

Quantity of reinforcement per m^3 of concrete:
7kg Barchip fibres

7.3 Project Benefits

- Off-site segment production: segments were produced at a local precast factory where synthetic fibre was mixed into the concrete then poured into vertical moulds. The initial segments were preassembled at the plant to form a trial ring to ensure that the correct tolerances were achieved. The segments were then horizontally stacked and trucked to site.
- The use of fibres in these segments proved very effective in meeting all the design requirements as well as ensuring that the segments sustained minimal damage from the jacking rams during installation. The segments have since performed to the specified design criteria.
- The use of fibres has lowered the overall carbon footprint of the project.
- Concerns over corrosion of the segment reinforcement were removed.

7.4 Picture Reference

Figure 29: Segmental lining of macrosynthetic fibre reinforced TBM tunnel

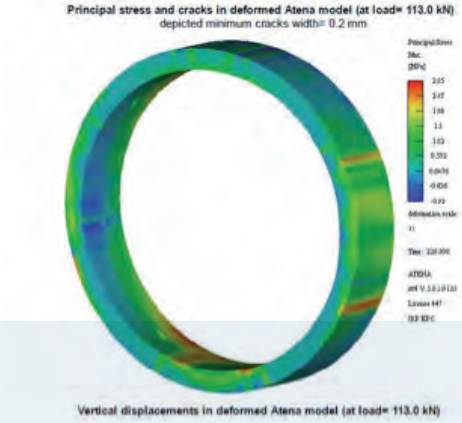

Figure 30: Finite element analysis designs by JKP Static

8. PROJECT REFERENCE CHANNEL TUNNEL RAIL LINK, LONDON

8.1 Project Details

History & brief description:
The UK's Channel Tunnel Rail Link (CTRL) between the Channel Tunnel Terminal at Cheriton and London rail terminal of St. Pancras was one of the largest construction projects in Europe at the time of construction. Fibre reinforcement was chosen for the segmental lining of the tunnels under London, based on a long history of successful steel fibre reinforced segmental linings in the UK.

Year: 2003-2007

Client and Location:
HS1 Ltd, UK

Type of tunnel:
Rail link

Ground conditions:
Soft rock, weak rock

Alignment length:
19km (single track twin tube tunnels).
Depth=25 – 50m

TBM type:
Operating thrust: 75000kN (30 shoes)

8.2 Design Approach Adopted

Specified strengths of concrete – compressive and residual tensile:
C50/60 (with low permeability specified to improve durability and minimise water ingress into the tunnel; Characteristic residual flexural strength=3,6 N/mm^2

Inner and outer diameter:
ID 7.15m, OD 7.85m

Ring segmentation:
9 segments +1 key

Dimensions of segments :
Thickness: 0.350m, ring length: 1.50m

Type of segment reinforcement:
Steel fibres & Micro polypropylene fibres

Quantity of reinforcement per m^3 of concrete :
30kg/m^3 Dramix RC 80/60 BN fibres & 1kg/m^3 PP fibres

8.3 Project key points

- Job site experience has shown that the time needed to install a 7 + 1 segment ring is almost the same as for 9+1 segments ring. Hence this segmentation was chosen.
- The segment thickness of 350 mm was driven by loading conditions. The design criteria for the tunnel linings were based on experience from other projects, including the Jubilee Line Extension in London and the MTRC in Hong Kong.
- The steel fibre reinforced concrete eliminated spalling at the contact surfaces, even in the case of bird's mouthing.
- Each segment mould has four vibrators and, after minor screeding, the segments are cured for six hours at 35 to 40°C in a chamber heated by hot water rather than steam.

8.4 Picture Reference

Figure 31: Demoulding of key segment

Figure 32: Segments stacked at the factory

9. PROJECT REFERENCE PISTA NUEVA MALAGA

9.1 Project Details

History & brief description:
A Joint Venture of Acciona Infrastructures and Sando Construcciones were awarded the construction of the suburban rail line by AENA. The extension to Malaga airport required an investment of €280 million. The Los Prados-Airport section runs almost entirely below the surface, crossing under the Guadalhorce River and Malaga Airport's new runway. The name of the tunnel is "Túnel Guadalhorce-Aeropuerto". It is an electrified double track tunnel and is part of "La línea C-1 de Cercanías Málaga", between Málaga and Fuengirola.

Client and Location:
ADIF (Administrador de Infrasrructuras Ferroviarias); Spain

Type of tunnel:
Railway

Ground conditions:
Alternating gravel, sand and clay layer

Alignment length:
2900m; Depth=15-28m

TBM type:
Operating thrust=8500-36600kN; Maximum theoretical thrust=83643kN

9.2 Design Approach Adopted

Design method & standard used:
Spanish code EH-91

Specified strengths of concrete – compressive and residual tensile:
50 N/mm^2, Flexural tensile strength= 5 N/mm^2, Residual Strength= 2.9 N/mm^2

Inner and outer diameter:
ID 8,43 m, OD 9,07m

Ring segmentation:
6 segments + 1 trapezoidal key

Dimensions of segments :
Segment Width: 1,5 m, Thickness: 0.320m,

Type of segment reinforcement :
Hybrid: macrosynthetic fibre & steel bar

Quantity of reinforcement per m^3 of concrete :
5 kg/ m^3 Barchip fibres & 98 kg/ m^3 bars

9.3 Project Benefits

Use of macrosynthetic fibres as non-corrosive reinforcement for:

- Crack control
- Reduced concrete cover
- Minimising spalling in handling and installation
- Bursting resistance under jacking loads

9.4 Picture Reference

Figure 33: Portal of the TBM tunnel

Figure 34: Stacked segments

The basic information given in the European Union's system of CE marking is the following:

- type of fibres: steel/polymer;
- CE Certification;
- type and dimension;
- tensile strength;
- Young's modulus;
- length;
- cross sectional form;
- diameter or dimensions of cross section;
- surface finish and anchorage (e.g. hooked at the end or embossed);
- tolerances on the length, the diameter (and the aspect ratio for steel fibres);
- safety aspects.

In addition the declaration of performance under standard tests is reported.

For CE-marking of fibres, two levels of attestation of conformity are defined: System 1 and System 3 – see Table 7.

System 1 is applicable when the fibres have a structural function, i.e. when the fibres are designed to contribute to the load-bearing capacity. The system requires a continuous surveillance of the production process of the fibres by an independent Certifying Body, which delivers a certificate of conformity (CE-mark).

System 3 is applicable when fibres are used for other reasons, i.e. for some non-structural function - for instance to reduce the risk of plastic shrinkage, or to improve the behaviour of concrete in fire. This system allows the manufacturer alone to declare that the quality is in accordance with the requirements of the standard: no confirmation by a third party is necessary.

In practice, therefore, when the post-crack strength of fibre concrete is taken into account in the structural design, the fibres must be certified under System 1, and the CE label on the packaging must indicate that the fibres are certified for structural use (System 1).

SYSTEM 1	SYSTEM 3
Field of use	
Structural use [A]	Non-structural use
Quality control	
• Initial type Testing (ITT) under the responsibility of the Notified certification Body • Initial and Annually Factory Production Control (FPC) assessment by Notified Body • Certification institute → "Certificate of Conformity"	• Initial Type Testing by a Notified Laboratory • Factory Production Control (FPC) under responsibility of the manufacturer • The manufacturer creates and signs a "Declaration of conformity"

Table 7: CE marking definition of sustem 1 and system 3

[A] Structural use of fibres is where the addition of fibres is designed to contribute to the load bearing capacity of a concrete element

>> Appendix F - example of a cost comparison

NUMBER OF RINGS IN PROJECT	SEGMENT / RING	m³ OF CONCRETE / RING	
3000	8	6,48	
1) STEEL FIBRE REINFORCED SEGMENTS			
	UNIT COST	m³ PER RING	COST/RING
C60 Concrete	84,00	6,48	544,32
Fibre dispenser	32000		10,67
		KG/m³	
Fibres *	1,3	40	336,96
Repair of damaged segments	Percentage	Segments for repair	
% of segments requiring major repairs	5%	150	
	MAN-HOURS/SEGMENT	COST/HOUR	
Labour for repair	1	14,00	0,70
Cost of craneage in/out repair areas	0,25	25,00	0,31
Administration/QC costs	0,25	14,00	0,18
	COST/SEGMENT	TOTAL COST	
Cost of repair materials	10,00	1500	0,50
Total Cost/Ring			893,63

Table 8: Exemple of a cost comparison

* macro synthetic fibres may be able to meet the design requirements of a project for a similar cost.

2) REINFORCEMENT CAGES			
	UNIT COST	m^3 PER RING	COST/RING
C60 Concrete	84,00	6,48	544,32
	UNIT COST / KG	KG/m^3	
Reinforcement	0,8	100,00	518,40
CAGE FABRICATION	MAN-HOURS/RING	COST/HOUR	
Labour for fabrication	16	12,00	192,00
Administration/QC costs	1	12,00	12,00
Jigs	25000		8,33
Rebar fabrication shed area	200000		66,67
Welding/tie wire	3000		1,00
PLACING OF CAGES INTO MOULDS	MAN-HOURS/RING	COST/HOUR	
Labour to place cages	3	10,00	30,00
Cost of cranage	3	25,00	75,00
QC costs (checking cover)	1,5	12,00	18,00
REPAIR OF DAMAGED SEGMENTS	PERCENTAGE	SEGMENTS FOR REPAIR	
% of segments requiring major repairs	10%	300	
	MAN-HOURS/SEGMENT	COST/HOUR	
Labour for repair	2	14,00	2,80
Cost of craneage in/out repair areas	0,5	25,00	1,25
Administration/QC costs	0,5	14,00	0,70
	COST/SEGMENT	TOTAL COST	
Cost of repair materials	10,00	3000	1,00
Total Cost/Ring			1471,47

Table 8 suite: Exemple of a cost comparison

INTRODUCTION

Following a succession of major tunnel fires across Europe, the need for effective fire protection for the structural concrete lining has become a matter of priority for both new and existing tunnels.

This document covers the "integral protection" incorporated into the structural concrete, based on the use of micro polypropylene fibres manufactured to EN 14889-2:2006 standards.

WHAT IS EXPLOSIVE SPALLING?

Concrete spalling can be described as the breaking off of layers or pieces of concrete from the surface of a structural element when exposed to the high and rapidly increasing temperatures experienced in fires. Three different kinds of concrete spalling can be categorized:

a) Surface Spalling

b) Corner Break-off

c) Explosive Spalling

Explosive Spalling is unquestionably the most serious and dangerous form of spalling. This occurs during the first 20 – 30 minutes of a fire when the temperature in the concrete is in the range of 150 - 250°C. Explosive spalling occurs when there is a rapid temperature rise, such as in hydrocarbon-fuelled fires following a traffic incident. Very large pieces of concrete can be violently ejected over several metres away from the concrete. As a fresh concrete face is presented to the fire, progressive explosive spalling continues deep into the concrete section, threatening the structural integrity of the construction by reducing the cross-section of concrete and exposing the reinforcement to fire. Steel rapidly loses its load bearing capacity from 250 °C onwards.

As a result of several decades of research it is known that there is a complex combination of chemical, physical and thermodynamic factors that influence explosive spalling, including moisture content, type and size of aggregate, concrete permeability, rate of heating, presence of reinforcement and external loadings (e.g. Sullivan 2001 and Khoury 2008). Experts agree that there is significantly more risk of explosive spalling when high strength, low permeability concrete is specified, because of the higher pore pressures which can build up during heating. The most critical parameters for explosive spalling are:

- Moisture content of concrete
- Density and strength of concrete (higher strength and density leads to more severe spalling)
- Gradient of temperature increase and peak temperature

The theories of how and why explosive spalling occurs are predominantly based upon moisture movement. As the temperature of the concrete increases, the moisture in the concrete changes to steam vapour. If it is unable to escape from the concrete matrix, this vapour creates a dramatic increase in pressure inside the concrete. As this process continues, the vapour pressure increases to the point where it exceeds the tensile capacity of the concrete, causing pieces of concrete to be violently and explosively dislodged from the element. As well as this conventional 'moisture movement' theory, there is also a consensus that aggregate expansion caused by thermal stresses also has a direct influence on explosive spalling. Despite ongoing research, a full understanding of the spalling and further damaging processes has not been found yet.

HOW DO THE POLYPROPYLENE FIBRES WORK?

It has been well accepted for many years that the addition of suitable polypropylene (PP) micro-fibres can counteract explosive spalling in cast concrete and sprayed concrete. Since explosive spalling is caused by the pressure created by a restriction on the movement of moisture/steam, then the presence of the fibres must relieve that pressure.

The simple explanation as to why PP fibres influence the reduction of explosive spalling is that when the fibre reinforced concrete is heated to approximately 160 °C the fibres melt to form channels that provide a pressure release mechanism within the concrete. The remains of the fibre are soot, occupying ca. 5% of the void. The newly formed cavity system reduces significantly internal pressures and the amount of explosive spalling that may otherwise occur.

FIBRE QUALITY

Fibres used for explosive spalling resistance should be PP micro-fibres (100 percent virgin polypropylene fibres, containing no reprocessed olefin materials) conforming to EN 14889-2:2006 Class 1a and specifically engineered & manufactured in an ISO 9001 certified facility for use as concrete secondary reinforcement. Where applicable, fibres should also carry the CE marking. The properties of the fibres should be constant. The variation of properties should be declared by the supplier. The PP micro fibres should have been tested and proven in accredited fire testing laboratories to show their effectiveness in reducing explosive spalling and suppliers should demonstrate a track-record of usage in tunnel lining applications. For the tests, the variability of the properties has to be taken into account.

Fibrillated PP fibres provide a limited degree of protection whilst macro synthetic and steel fibres have been found to have little or no influence on the prevention of explosive spalling, unless the moisture content of the segments is low (i.e. less than 3%).

FIBRE DOSAGE

The concrete specification and the fire risk assessment are important factors to consider when selecting the dosage of PP micro-fibres to use for passive fire protection. The accurate determination of the actual minimum fibre dosage, to provide the required explosive spalling resistance, can only really be established by large scale

fire testing of the actual concrete which is to be used on a specific project. Large scale means to do the test under same load, stress and fixity conditions as the concrete will have at the construction site

This is a costly exercise and an expense that many projects would like to avoid. Section 6.2 of EN 1992-1-2:2004 makes reference to the use of 2 kg/m3 of monofilament polypropylene micro-fibres to control explosive spalling in high strength concrete. The properties of fibres (type, diameter, length) and strength of concrete which relate to this recommendation are not defined. Many engineers follow this recommendation to eliminate the need for expensive testing in the knowledge that this dosage will provide a good safety margin. This does not preclude the usage of lower dosages but it does highlight the need for careful consideration and a necessity to carry out fire testing on large concrete samples that completely replicate the materials to be used on an actual project. Where this has been done, dosage rates of, for example, 1.0 kg/m3 and 1.5 kg/m3 have been used in actual tunnel projects. When conducting fire tests also the strength and density development of concrete should be taken into account.

PRACTICAL CONSIDERATIONS

During the selection process for PP micro-fibres, it is imperative that designers take into consideration the effect of the fibres on the rheology of the concrete, e.g. concrete workability, air content and strength. This can present some difficulties in the casting process. This optimisation of the mix and process should be done prior to any fire testing procedures so that potential changes to the mix design due to the fibre addition are eliminated. This will ensure that the positive and negative effects of fibres, together with any additional cement or admixture costs and "in-place" concrete costs, are taken into account so that the various options can be correctly compared.

The addition of PP micro-fibres to concrete is a relatively simple process depending on the size of the project. Fibres designed specifically for concrete reinforcement are normally supplied in fully degradable paper packaging that enables the desired dosage per unit volume to be simply added directly into the concrete truck or pan mixer. The packaging is designed to rapidly break down allowing uniform distribution of the fibres into the concrete. In relatively small projects this is often the most cost effective method to adopt, with packaging available in 1 kg or 2 kg bags. Where projects involve significant quantities of PP micro-fibres, the contractors and ready-mixed suppliers often consider the use of more sophisticated, automated and integrated dosing systems for adding fibres to the concrete. The mixing time has to be determined to guarantee a homogeneous mixing of PP-fibres.

CHOOSING THE CORRECT FIRE CURVE FOR YOUR PROJECT

There are a number of fire time-temperature curves proposed for a variety of applications, ranging from the ISO 834 (1975) 'cellulosic' curve to the RWS curve (Figure 35). The selection of an appropriate time-temperature curve and the relevant fire duration is an important consideration, which should be driven by a risk assessment, in cases where no design standards apply.

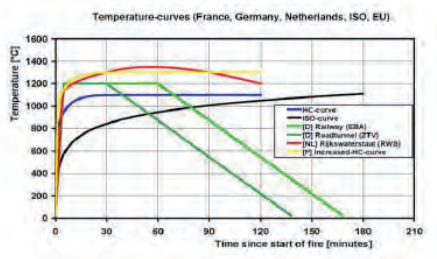

Figure 35: Fire time-temperature curves (ITA 2004)

Ingason (2006) reported on the UPTUN series of experimental fire tests on a disused tunnel, and made recommendations regarding the most appropriate fire temperature curve for a range of fire risks (Table 9). The ISO 834 curve is recommended up to an expected fire heat release rate of 50MW, above which the hydrocarbon curve (up to 100MW) and thereafter the RWS curve (up to the stoichiometric limit) should be applied. However, these recommendations should be considered as preliminary only, since they are not yet backed up by sufficient evidence. EFNARC (2006) provides further information on this subject.

Once a suitable fire time-temperature curve has been selected, the likely effects of such a fire on the tunnel's structure should be ascertained. EN 1992-1-2:2004 provides methods of calculating the reduction of concrete strength due to high-temperature damage within the concrete and its steel reinforcement. EN 1992-1-2:2004 also provides guidance on the reduction in the cross-section due to fire damage, based on cellulosic fires as per ISO 834. The calculation of the structural response to non-ISO 834 fires within concrete members is currently not covered by European standards.

CONCLUSIONS

For more than 14 years, PP fibres have been used in millions of cubic metres of concrete worldwide to help reduce the incidence of explosive spalling in concrete. During this period the understanding of the mechanisms, by which these materials provide resistance to explosive spalling, has developed, although it is fair to say that some experts contend that the definitive mechanisms are yet to be identified.

If PP micro-fibres are to be used to provide explosive spalling resistance in concrete tunnel linings, the chosen mix design should satisfy the following criteria:

- Demonstrate ability to counteract explosive spalling
- Avoid any negative side effects on the concrete mix and placement.

The message that PP micro-fibres reduce explosive spalling is clear to lots of engineers and clients but it is important not only to choose the correct fibre type but also the correct aggregate/concrete mix and placement process.

SIZE OF FIRE IN MW			EXAMPLES OF ROAD VEHICLES	EXAMPLES OF RAIL VEHICLES	EXAMPLES OF METRO VEHICLES	FIRE CURVE
RISK TO LIFE		5	1 -2 cars			ISO 834
		10	Small van 2 -3 cars	Electric locomotive	Low combustible passenger carriage	ISO 834
		20	Big vans, public buses, multiple vehicles		Normal combustible passen-ger carriage	ISO 834
	RISK TO STRUCTURAL INTEGRITY	30	Bus, empty HGV	Passenger carriage	Two carriage	ISO 834
		50	A combustible load on a truck	Open freight wagons with lorries	Multiple carriages >2	ISO 834
		70	HGV with a combustible load (approx. 4 tonne)			Hydro carbon
		100	HGV average			Hydro carbon
		150	Loaded with an easily combustible load (approx. 10 tonnes)			RWS
		≥ 200	Limited by oxygen, petrol tanker, multiple HGV's	Limited by oxygen		RWS

Table 9: UPTUN fire resistance recommendations (Ingason, 2006)

This section should be read in conjunction with the section 3.3.3.3.

SAMPLE PREPARATION

The test specimens shall be prisms conforming to EN12390-1 with a section of 150±0.75mm (between the moulded surfaces) x 150±1.5mm (between trowelled and bottom moulded face), and a length between 550 and 700 mm. A length of 600 mm is often used in practice. Two methods can be used to fill the moulds.

Preferably, the specimens are made under lab conditions. In this case, a pan mixer shall be used to mix the concrete, and all the ingredients must be weighed accurately. Aggregates, sand and cement are added to the mixer (in this order) and mixed for 60 seconds. Then the water is added, and the concrete is mixed for another 60 seconds. After this step, the fibres are added, and mixed for 270 seconds to make sure that all fibres are separated and homogeneously distributed. The mixer can then be stopped, and a visual inspection of the distribution of the fibres needs to be performed.

Filling the moulds is done in one action, up to a height of 110 % of the mould (see Figure 36). The concrete is then vibrated on a vibrating table (unless it is self-compacting concrete), and levelled off during vibration (see Figure 37). Full compaction is achieved when no further appearance of large air bubbles on the surface of the concrete can be seen, and the surface is becoming relatively smooth with a glazed appearance, without excessive segregation. Internal vibration is not recommended so as not to disturb the 3-dimensional homogeneous distribution of the fibres. If a vibration needle is used, the zone around the vibration needle will contain fewer fibres than the rest of the beam.

Figure 36: Filling the mould of an EN14651 beam

Figure 37: Vibrating an EN14651 beam

If no laboratory is available, the alternative method described below (see Figure 38) should be taken into account to fill the moulds. The size of increment 1 should be twice increment 2. The mould shall be filled up to approximately 90 % of the height of the test specimen before compaction. The mould shall be topped up and levelled off while being compacted. Compaction shall be carried out by external vibration. In the case of self-compacting FRC, the mould shall be filled and levelled off without any compaction.

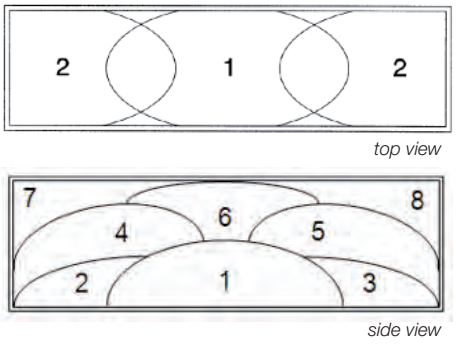

Figure 38: Alternative method to fill moulds for EN14651 beams

Curing of the test specimens is the same as for the EN14488-5 square panels, and shall be done according EN12390-2. They should be left in the moulds for at least 16 hours, but no longer than 3 days, protected against shock, vibration and dehydration at a temperature of 20±5°C. After removal from the mould, the specimens shall be rotated, if necessary, and then sawn through the specimen width at mid-span (see Figure 39 for orientation of notch relative to top surface during casting).

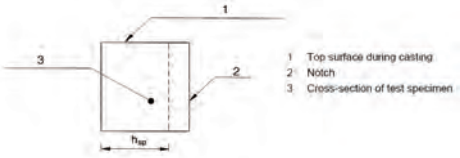

Figure 39: Position of the notch in an EN14651 beam

The width of the notch shall be 5 mm or less, and the distance of hsp, mentioned in Figure 35, shall be 125±1mm.

The test specimens shall be cured for a minimum of 3 days after sawing until minimum 3 hours before testing, water at a temperature of 20±2°C, or in a chamber at 20±2°C and a relative humidity of minimum 95%. Regular checks should be made that the surfaces of the specimens in the chamber are continuously wet.

Loss of moisture and deviations from the required temperature should be avoided at all stages of transport by, for example, packing the hardened specimens in wet sand, wet sawdust or wet clothes, or transporting them in sealed plastic bags containing water.

TEST PROCEDURE

Normally, testing shall be performed at 28 days.

Testing of the specimens is done in a 3-point bending test, but can be performed in two ways. In the first method, the crack (or notch) mouth opening displacement

(CMOD) is measured, and a displacement transducer is mounted along the longitudinal axis at the mid-width of the test specimen. The distance between the bottom of the specimen and the line of measurement shall be less than 5 mm (see Figure 40).

A second possibility is to measure the deflection instead of the CMOD. In that case a displacement transducer shall be mounted on a rigid frame that is fixed to the test specimen at mid-height over the supports. One end of the frame should be fixed to the specimen with a sliding fixture, and the other end with a rotating fixture. A thin plate fixed at one end can be placed at mid-width across the notch mouth at the point of measurement (see Figure 41).

The tests are preferably deflection controlled. To control the test with a CMOD, knifes need to be glued next to the notch. It is possible that they come loose during the test due to a bad connection between the knives and the concrete. It is easier to mechanically fix the deflection transducer to the concrete specimens. Fewer test specimens and test results will be lost in this way.

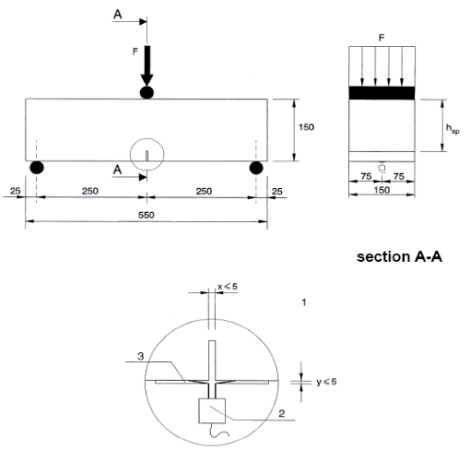

Figure 40: EN14651 test setup with CMOD transducer

The testing machine should be capable of operating in a controlled manner, producing a constant rate of displacement (CMOD or deflection), and have a sufficient stiffness to avoid unstable zones in the load-CMOD curve or the load-deflection curve. A total stiffness of the system of 200kN/mm (including frame, load cell, loading device and supports) is advised as a minimum requirement.

All rollers should be made of steel and have circular cross section with a diameter of 30±1mm. Two of the rollers, including the upper one, shall be capable of rotating freely around their axis and of being inclined in a plane perpendicular to the longitudinal axis of the test specimen. The distance between the centres of the supporting rollers shall be equal to 500±2 mm.

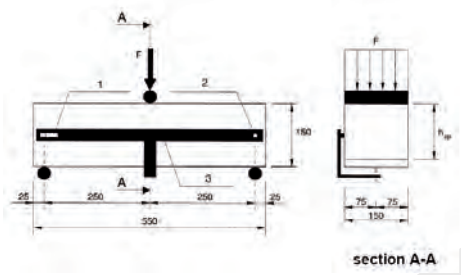

Figure 41: EN 14651 test setup with deflection transducer

The load measuring device needs an accuracy of 0.1 kN and the linear displacement transducer needs an accuracy of 0.01 mm. The data recording system should be able to record load and displacement at a rate not less than 5 Hz.

For the case of a testing machine controlling the rate of increase of CMOD, the machine shall operate from the start of the test with a CMOD-increase of 0.05 mm/min and datalogging at minimum 5 Hz. When CMOD = 0.1 mm, the machine shall operate at a CMOD-increase of 0.2 mm/min and a minimum datalogging of 1 Hz. The test shall not be terminated before a CMOD value of 4 mm.

For the case of controlling the rate of increase of deflection, the machine shall start the test with a deflection increase of 0.08 mm/min with a datalogging of minimum 5 Hz. When the deflection reaches 0.125 mm, the deflection increase shall be changed to 0.21 mm/min until a final deflection of 3.5 mm, and a data-logging of minimum 1 Hz.

If the crack starts outside the notch, the test result should be rejected.

TEST VALIDATION AND QUALITY CONTROL

Bending tests must be performed on a minimum number of specimens and the variation of the results in terms of residual strength, fR,j, must not exceed the following values:

- Characterization tests: 12 no. specimens and residual strength variation ≤ 25%
- Control tests: 9 no. specimens and residual strength variation ≤ 25%

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