The use of Steel and Synthetic Fibres in Concrete under Extreme Conditions

Don Wimpenny[1], Wolfgang Angerer[2], Tony Cooper[3] and Stefan Bernard[4]
[1]Principal Materials Engineer, Halcrow Pacific Pty Ltd
[2]Senior Tunnel Engineer, Halcrow Pacific Pty Ltd
[3] Consultant Elasto Plastic Concrete
[4] Consultant, TSE Pty Ltd

Synopsis: The tunnelling and maritime environments can provide extreme conditions for reinforced concrete. Exposure to aggressive saline water can be combined with design lives of 100 years or more. In recent years, traditional carbon steel reinforcement has been substituted by the use of steel fibres and, more recently, by synthetic fibres for sprayed concrete and precast items, such as tunnel segments. This paper describes some of the factors which have prompted the change from rebar to fibres and includes a detailed consideration of the specific durability design aspects of fibre reinforced concrete use within desalination facilities.

Keywords: fibres, synthetic, steel, design, durability, tunnels, desalination.

1. Introduction

The use of fibres to enhance the properties of construction materials can be traced back over 4000 years to the use of straw in bricks and horse hair in plaster. Fibres can reduce plastic cracking in fresh concrete and enhance the post-crack ductility of hardened concrete. The elimination of reinforcement fixing can have significant time, cost and safety benefits. Whilst the random orientation and dispersal of fibres means they are not as efficient as conventional reinforcement for dealing with predictable stresses, they are able to resist crack propagation under unforeseen stresses, particularly those arising close to the surface of elements during construction and in service, such as impact.

Fibres can be particularly beneficial under extreme environments, such as exposure to chlorides and fire. Fire and abrasion resistance are enhanced and the discrete nature of fibres means that the risk of corrosion and associated spalling is significantly reduced.

The mining and tunnelling industry makes extensive use of fibres in sprayed concrete linings for underground support. Fibres allow the lining to retain ductility, even under high deformation, which is critical for safety. Precast tunnel segments have utilised both steel and synthetic fibres for handling and improved fire resistance respectively (1). Marine works have used synthetic macro fibres to eliminate corrosion risk under exposure to seawater (2).

This paper provides a review of structural and durability design approaches based on published guidance. It should be noted that performance is dependent on the particular fibre type, including the manufacturer.

2. Fibre reinforced concrete

2.1 Fibre types

The principal fibre types in BS EN 14889 (3,4) are shown in Figure 1.

The shape and surface texture of the fibres is important in determining their effect on concrete properties. Fibres should fail by gradual pull-out after disbondment from the cement matrix in order to provide ductility and features such as hooked ends, crimping, twisting and embossing are intended to promote friction associated with the pull-out mode of failure.

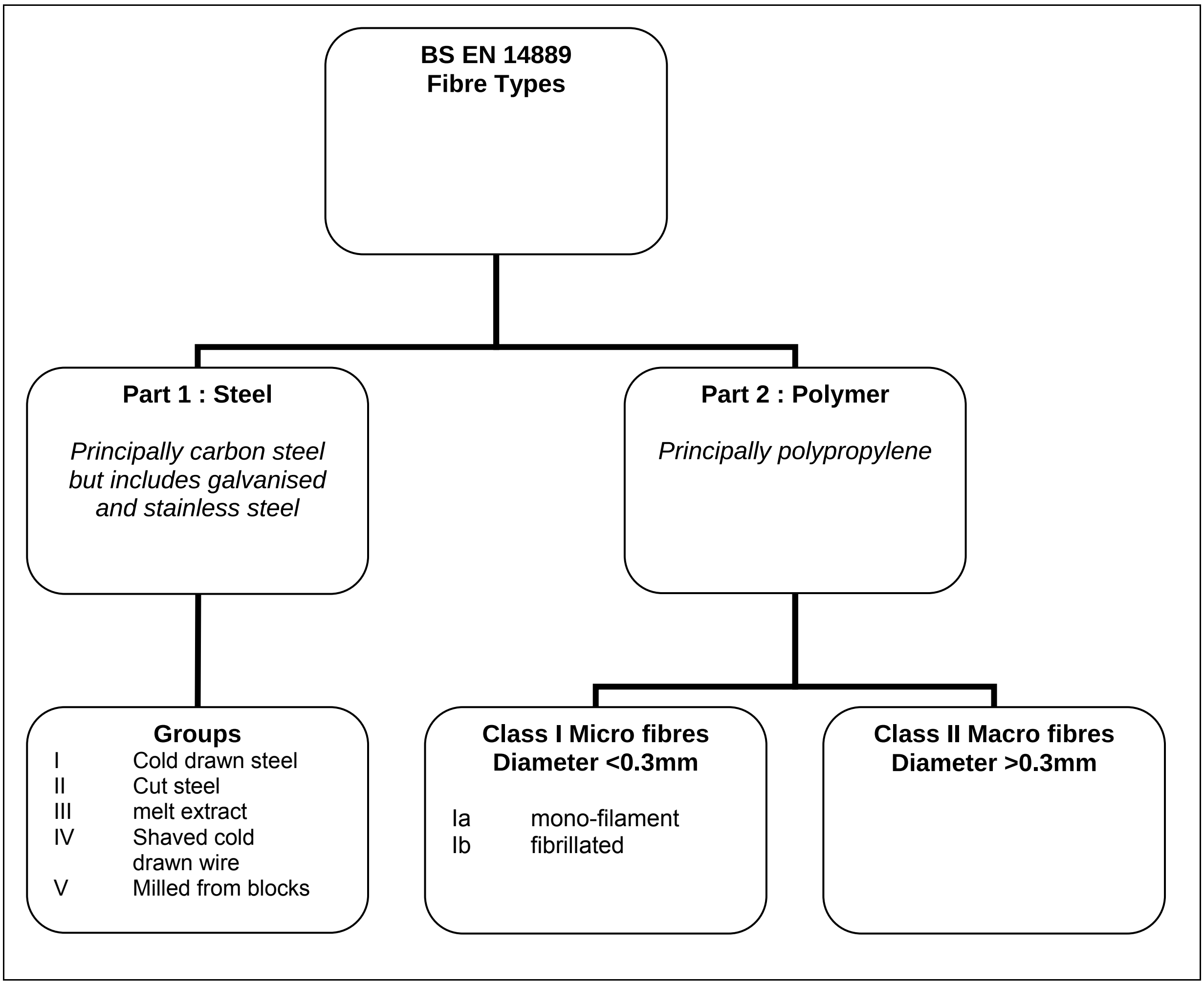


Figure 1. Fibre types to BS EN 14889.

Table 2 summarises the characteristics of two widely used fibre types and compares the attributes of concrete made with these fibres to conventional concrete.

Short term laboratory and field studies on the durability of uncracked steel fibre reinforced concrete (SFRC) indicate carbonation induced corrosion is restricted to those fibres immediately below the surface (5). Galvanic corrosion and spalling do not appear to occur (6), but there is a potential for corrosion at cracks (7). This is important, as it could ultimately lead to sudden failure of the concrete due to fibre breakage rather than ductile failure by fibre pull out (5). It is recommended that limiting crack width of 0.1 to 0.2mm be adopted, depending on the service conditions.

Corrosion of steel fibres will cause staining and where this would be unacceptable, galvanized steel is sometimes used (as the solubility of zinc is increased when chloride ions are present, stainless steel and synthetic macrofibres are more appropriate in a chloride-rich environment).

Structural synthetic fibres are an alternative to steel fibres for controlling handling damage and providing long-term ductility. They are not significantly affected by exposure to seawater (8) or sodium chloride at temperatures of 20-40°C (9). However, the maximum service temperature should ideally be limited to 60°C and exposure to some chemical agents, such as chlorine gas, should be avoided.

Table 2. Comparison of steel and synthetic fibres.

Characteristic	Steel Fibres	Synthetic micro-fibres	Synthetic macro-fibres
Characteristic of Fibres			
Shape/Texture	Cold drawn hooked ends	Straight smooth	Continuously embossed
Collation	Glued bundles	Fibrillated	Uncollated
Length (mm)	60	12	48
Diameter (mm)	0.75	0.02-0.03	0.5-1
Tensile Strength (MPa)	1050	30	550
Elastic Modulus (GPa)	>200	2	10
Dosage (kg/m^3)	25-35	1-2	6-10
Service temperature (°C)	300	60	60
Melting point (°C)	>800	150	150
Base material	Carbon steel	Polypropylene	Polyolefin (polypropylene, polyethylene)
Comparison with conventional concrete (Unreinforced except where indicated by*)			
Workability	Reduced	Slightly reduced	Slightly reduced
Plastic shrinkage cracking	Unaffected	Reduced	Slightly reduced
Early-age thermal cracking	Reduced	Unaffected	Reduced
Long-term shrinkage cracking	Reduced	Unaffected	No data
Stray current corrosion	Reduced	Unaffected*	Eliminated
Durability in chloride exposure*	Increased	Unaffected*	Greatly Increased
Fire spalling resistance	Slightly Increased	Greatly Increased	Increased
Compressive strength	Unaffected	Unaffected	Unaffected
Residual flexural strength	Increased	Unaffected	Increased
Impact strength	Greatly Increased	Unaffected	Increased
Flexural toughness	Increased	Unaffected	Increased
Abrasion resistance	Increased	Slightly increased	Slightly increased
Freeze-thaw resistance	Slightly increased	Increased	Increased
Flexural energy absorption	Greatly Increased	Unaffected	Greatly Increased
Concrete permeability	Slightly increased	Slightly increased	Slightly increased
Pump wear	Increased	Reduced	Reduced
Safety*	Hazard from handling and protruding fibres	Increased	Increased
Finishing	Extra care during floating	Exposed fibres soon abrade	Fibres may float and protrude in poorly designed mixes

2.2 Field examples

Evidence of satisfactory performance of fibres in concrete elements is important in providing confidence to potential users. Unfortunately, there is relatively limited information on long-term durability of FRC in the field. For example, steel and synthetic macrofibres have only been employed in sprayed concrete since the 1970s and the 1990s respectively.

Table 3 summarises a selection of projects where the concrete is exposed to extreme service conditions. Despite the many projects utilising fibres, the duration in service is modest compared to the design lives of 100 years, or more, which are increasingly required.

Table 3. Examples of fibre reinforced concrete under extreme conditions.

Project	Country	Application	Fibre type	Date entered service
Halsney Tunnel Project	Norway	Sprayed concrete lining to a sub-sea tunnel	Macro-synthetic	2005
Atlantic Ocean Tunnel Project	Norway	non-corrodible reinforcement in a sub sea tunnel	Macro-synthetic	2009
E18 Motorway	Norway	Sprayed concrete lining to tunnel close to waterfront	Steel	2009
E18 Motorway	Norway	non-corrodible reinforcement in ground with high sulphide levels	Macro-synthetic	2009
Sydney Northside Storage Tunnel	Australia.	sprayed concrete lining under chloride exposure	Macro-synthetic	2003
Docklands Light Rail Extension	United Kingdom	track slab subject to stray currents and sulfate and chloride exposure	Macro-synthetic	2004
Quarry Bins	United Kingdom	Sprayed concrete repair to abrasion damaged aggregate storage bins	Steel	1998
Blackpool South Shore	United Kingdom	Precast revetment units subject to handling, abrasion and seawater	Macro-synthetic	2006
Channel tunnel rail link	United Kingdom	precast segments subject to fire loading	Steel and micro-synthetic	2003
Gold Coast Desalination Project	Australia	Segmental lining for intake and discharge tunnels	Steel	2008

3. Structural design

3.1 Design methodology

The design of an FRC section follows the same approach used for the design of reinforced concrete, with appropriate modifications for material properties in the tension zones. In reinforced concrete the properties of the concrete and reinforcement can be determined separately. For FRC the properties of the composite of fibre and concrete, such as tensile splitting strength, flexural strength at first crack and residual flexural strength, should be determined concurrently. These properties can utilised in standard equations, which define the load capacity of the composite element.

Initial designs may use manufacturer's data or data from previous projects to estimate the material properties. However, it is important that the validity of the data is checked and that the performance of the material is demonstrated by testing in pre-production trials and by regularly sampling throughout production.

Figure 2 shows the stress-strain diagram adopted within the Rilem σ-ε design method. The values of stress and strain in the diagram are defined in RILEM TC-162 TDF (10) and the design standards using this approach such as Eurocode 2 (11) or NZS3101 (12). Similar recommendations are given in guidance by the German Concrete Association and in Concrete Society Technical Reports 63 and 65 (13, 14).

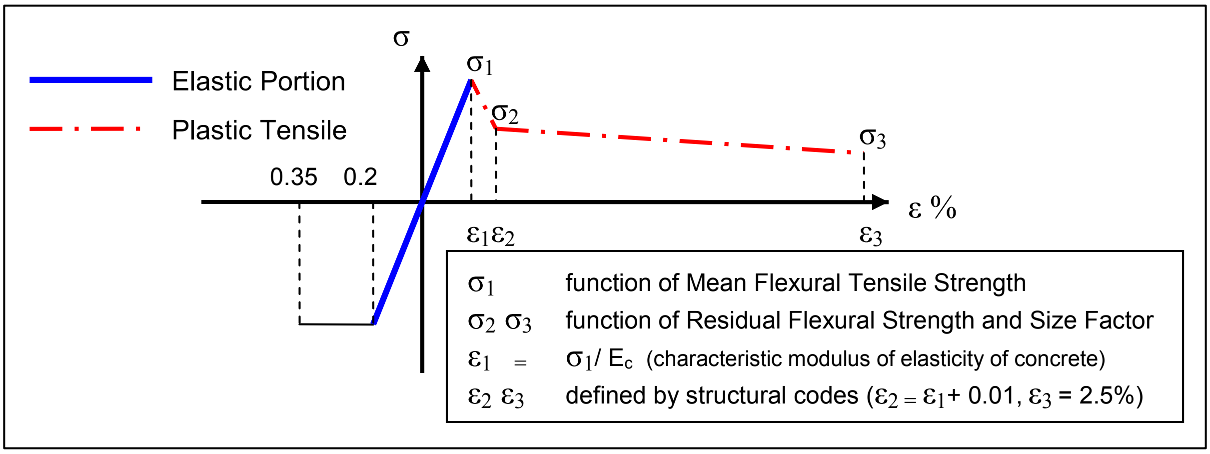


Figure 2. Stress-strain diagram for fibre reinforced concrete.

Although codes and recommendations mainly provide information regarding steel fibre reinforced concrete, the design principles apply equally to other structural fibres. Testing may be required to confirm the validity of some of the design factors and allowance should be made for the difference in stress-strain behaviour, long-term performance and implications of exceptional load scenarios, such as fire.

The properties determined from specific tests will differ from the full-scale structure. The designer should verify that properties are based on representative concrete samples and provide adequate allowances for the variability of the test method, the pattern of loading and the difference in geometry between the test specimen and the actual structure. Care should be taken in the choice of test method (eg. beam or panel) and specified limit. In particular, high residual flexural strength requirements at high deflection values may have no relevance to the design and prove unnecessarily difficult to achieve.

The full life of the structure should be considered in the structural deign (5). For example, bursting performance of tunnel segments during construction is based on tensile splitting and compressive strength. Permanent loading for the serviceability limit state is normally based on ensuring elastic un-cracked behaviour and as such relies on the flexural strength at first crack. The ultimate limit state allows for some plastic behaviour and utilizes the residual flexural strength. However, fibres are generally not relied upon as the primary reinforcement in plastic hinge regions. Structural designs for serviceability generally incorporate limits on stress, strain (eg. 10 microstrain in the tensile section) or crack width (eg. 3mm maximum for durable fibres).

Due to the random distribution of the fibres in a concrete matrix, there is no recognised method to calculate crack widths in fibre reinforced structures. A reliable first approximation of crack widths can be estimated in members subject to axial and bending forces by derivation of stress-strain diagrams in a linear elastic analysis. Crack widths can be therefore be predicted according to the following equation:

$$w = \varepsilon_b (h-x) \qquad (1)$$

where w is the crack width, ε_b is the strain at the extreme tension fibre, h is the section thickness and x is the depth of the compression portion (Figure 3).

However, the values obtained from such assessments are often higher than the 0.1 to 0.2mm limit for steel fibres in extreme exposure conditions (Section 2.1). Furthermore, the design methods are not reliable enough to ensure these crack widths will be consistently achieved. A prudent design approach for SFRC exposed to chloride from seawater or de-icing salts is to prevent the section from cracking during serviceability loading.

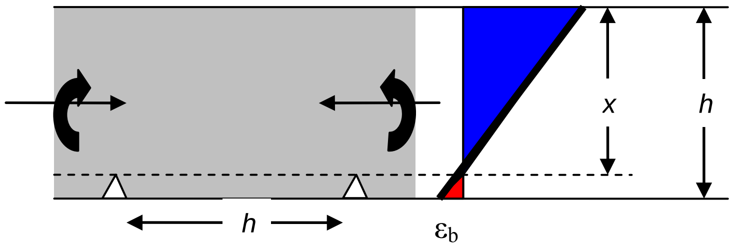


Figure 3. Calculation model for estimation of crack width in fibre reinforced members subject to axial compression and bending

Long-term performance of FRC is heavily influenced by the interaction between the fibres and the concrete matrix. Two aspects are particularly important. Firstly, the aim of the addition of fibres is to avoid the brittle failure associated with unreinforced concrete. In FRC the development of excess strength and hardness in the enveloping concrete matrix may result in a change from ductile behaviour to brittle behaviour, as fibres fracture instead of gradually pulling out of the matrix (15).

Secondly, the discontinuous nature of fibres means that they may exhibit slip within cracked concrete leading to creep under sustained loading. Recent research by Bernard on the creep of cracked FRC (16) indicates that creep depends on the fibre type, dosage and applied load (Figure 4). The results are specific to the particular fibre types tested. The designer needs to verify the influence of creep behaviour on the structure and provide appropriate contingencies in the design, such as limiting the tensile stresses.

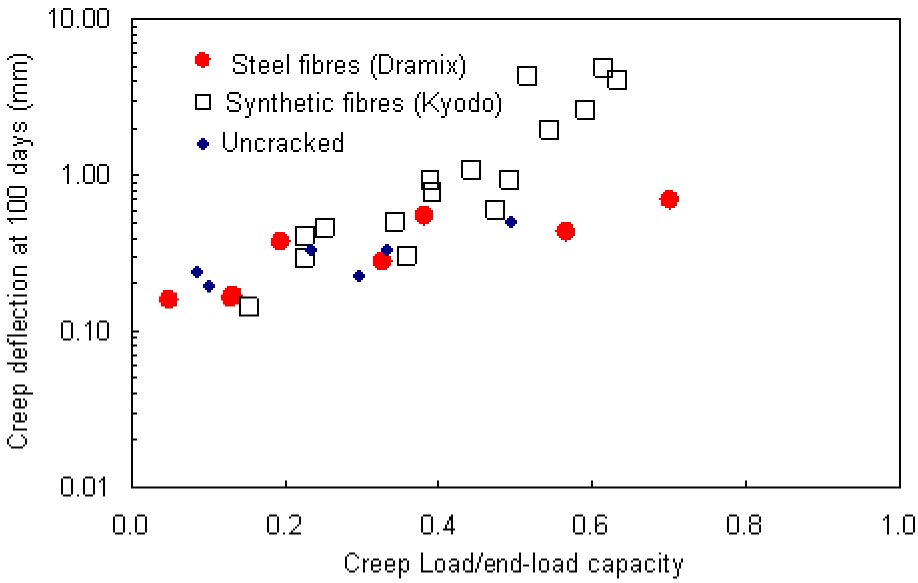


Figure 4. Relationship between creep deflection at 100 days and imposed load for ASTM C1550 panel test (16).

4. Durability design

4.1 Design methodology

The durability design for steel fibre reinforced concrete involves the prediction of carbonation and chloride ingress and the associated corrosion. In conventionally reinforced concrete, corrosion damage to the concrete, such as spalling or cracking, would often be used as the serviceability limit state. In the case of FRC, the loss of cross-sectional area of fibres by corrosion determines the depth of concrete over which the contribution of fibres to tensile and flexural strength should be ignored by the structural designer. A limiting value of 20% loss is typically used.

The accuracy of the assumed exposure conditions is critical to the validity of the predictions. In the case of a desalination facility, conditions during construction and any maintenance outages should be considered, as these may be more severe than those in service. Table 4 indicates the input parameters defining the service conditions and the characteristics of the concrete mix.

Table 4. Input parameters to durability modelling

General input parameter	Specific input parameters
Relative humidity (%) Temperature (°C) 28-day cube strength (MPa) Cementitious content (kg/m^3) Tricalcium aluminate level of cement (%) Slag, fly ash or silica fume level (%) Reinforcement diameter (mm) Fibre thickness (microns) Cover (mm) Allowable corrosion loss (%) Service life (years)	**Carbonation:** Carbon Dioxide Level (%) Intrinsic oxygen Permeability (m^2) Curing (days) **Chloride:** Background and surface chloride levels (%) Chloride diffusion value (m^2/s) and age (days) Aging factor Exposure type (wetting/drying, submerged, atmospheric) Steel type (stainless, carbon) Additional protective measures (coatings, corrosion inhibitor)

The surface level of chloride will vary depending on the salinity in the environment and amount of contact the concrete has with the environment. For concrete in the marine environment the surface chloride level will typically vary from 1-2% for submerged and atmospheric exposure to 4% for wetting and drying. For desalination schemes where the salinity can be 50% higher than seawater surface chloride levels may be correspondingly increased.

The carbonation and chloride models used in durability design are discussed below.

4.2 Carbonation model

The carbonation model commonly used is based on work by the Building Research Establishment (17) showing a good correlation between the carbonation rate and oxygen permeability. The latter is predicted based on the cement type, concrete strength and duration of curing. Gas permeability or diffusion testing should be carried out on proposed concrete mixes to validate the assumed oxygen permeability value.

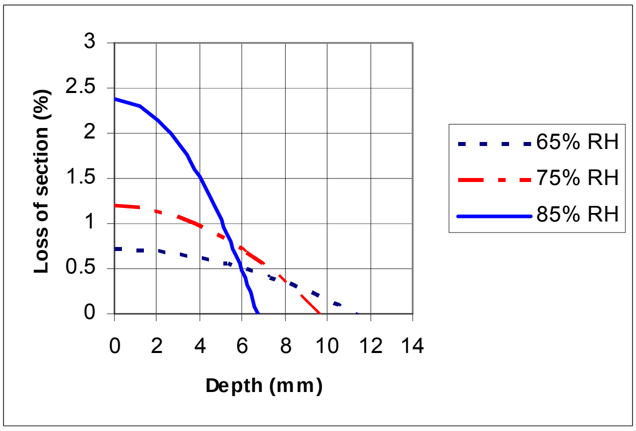
The modelling is normally deterministic rather than probabilistic, in that it adopts single values for each parameter rather than allowing for variation and then predicting the likelihood of different outcomes. The adopted values often reflect the worst likely conditions for temperature and carbon dioxide levels. The carbon dioxide level will typically lie between 0.04%, the average outdoor value, and 0.1% for poorly ventilated spaces (17).

Relative humidity is a key parameter affecting carbonation and a sensitivity analysis should be carried out to encompass the range of possible values during construction and in service.

Typical output from the model is shown in Figure 5.

Summary of Input		
Parameter	**units**	**value**
relative humidity	%	85
temperature	°C	25
carbon dioxide level	%	0.03
28-day cube strength	MPa	50
cementitious content	kg/m^3	400
ggbs level	%	0
pfa level	%	30
minimum cover	mm	30
bar diameter	mm	16
fibre thickness	micron	750
acceptable loss of section	%	20
curing	days	3
oxygen permeability	$10^{-16}m^2$	1
service life	years	100
Summary of Output (rebar)		
corrosion initiation	**52** years	
propagation of 0.1mm cracks	**3** years	
propagation of 0.3mm cracks	**15** years	
service life to 0.1mm cracks	**55** years	
cover for service life	**41** mm	
service life to 0.3mm cracks	**67** years	
cover for service life	**37** mm	
Summary of Output (fibres)		
depth of loss of section of 20%	**32** mm	

(a) Summary of input and output



(b) Graphical output showing loss of fibres during construction period

Figure 5. Typical output from carbonation model

4.3 Chloride model

The chloride ingress by diffusion is predicted using a model based on published research (18). The model uses Fick's second law of diffusion:

$$C_x = C_{sn}\left(1 - erf\frac{x}{2\sqrt{D_c \times t}}\right) \quad (2)$$

where D_c is the diffusion coefficient (m^2/s), C_x is the chloride concentration at depth x (m) after exposure time t (s), C_{sn} is the notional surface chloride level (% by mass of binder) and *erf* is the error function. The diffusion coefficient will increase with temperature but reduce with age, dependent on the concrete mix and the exposure conditions. The model allow for these effects, as well for as the release of chlorides due to carbonation (18). The chloride diffusion coefficients assumed in the design should be verified by appropriate testing, such as the NTB 492 method (19).

The chloride levels increase until the threshold chloride level is surpassed and then corrosion is initiated. The threshold level depends upon the cementitious type, service temperature, presence of corrosion inhibitors and reinforcement type (eg. carbon steel, stainless steel).

The rate of corrosion of steel is assumed to increase as the chloride level in the concrete adjacent to the steel increases. For example, the tidal and splash zone in marine conditions:

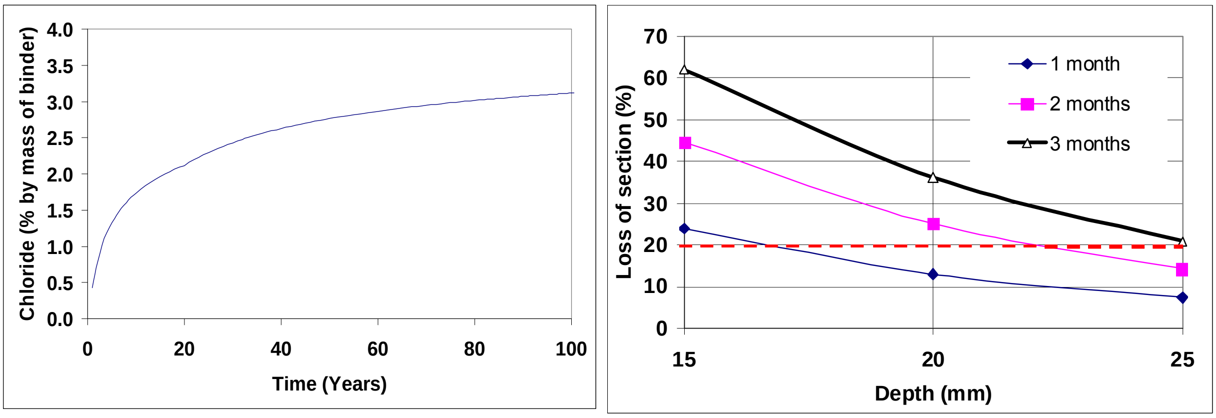
$$CR = 0.46\left(e^{1.84Cx}\right) \quad (3)$$

where CR is the corrosion rate (microns/year) and C_x is the chloride concentration (% by mass of binder) at the depth of the steel reinforcement.

The corrosion rate will depend on the temperature and relative humidity. An adjustment for temperature can be made using Arrhenius law (18).

The model allows different corrosion mitigation measures to be assessed, including: changes to the concrete mix and the use of protective coating, integral waterproofers, silane, controlled permeability formwork and stainless steel reinforcement.

The cumulative corrosion from each stage of the structures life is calculated and used to estimate a depth over which the effect of the fibres should be ignored, in a similar way to the carbonation. Figure 6 shows typical graphical output from the modelling.



(a) chloride ingress 50mm depth

(b) loss of section of fibres for outages of different duration

Figure 6. Output from chloride model

5. Conclusions

Fibres provide significant advantages over reinforced and plain concrete under extreme conditions, including exposure to fire, abrasion and seawater. Steel fibres and synthetic macrofibres increase post-crack ductility. This is an important consideration for use in ground support,

Steel fibres by virtue of their discrete nature and small diameter appear to eliminate galvanic corrosion and associated spalling damage compared to steel rebar and enhance resistance to chloride and carbonation induced corrosion. However, maximum crack width should be limited to 0.1-0.2mm depending on the service life and exposure conditions. Synthetic macrofibres are non-corrodible but will be affected by elevated temperature and some chemical agents.

Structural design methods are available for fibre reinforced concrete. These should use values of compressive, flexural and tensile strength based on performance tests on the FRC and should consider the whole life of the element. Serviceability limit state design should generally be based on elastic behaviour of an uncracked section using flexural strength to first crack data. Ultimate limit state design can allow for some plastic behaviour using residual flexural strength data.

The interaction between the concrete matrix and the discrete fibre is critical. SFRC may lose ductility when the concrete matrix becomes too strong leading to fracture rather than pull-out the fibres. Creep due to slip and elongation of fibres is also an issue. Deformation and creep in cracked FRC depends on fibre type, dosage and loading.

Durability design of facilities, such as desalination schemes, should assess the conditions during construction, operation and at times of outages. Carbonation and chloride ingress can be predicted from models based on published research. The loss of cross-section of fibres by corrosion can be used to estimate the depth of concrete over which any effect of fibres on flexural or tensile strength should be ignored.

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