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TIGHT MANOEUVRES

High-pressure grouting research for tunnelling

A SERVICE LIMIT STATE DESIGN APPROACH FOR SFRC TUNNEL LININGS

Sotiris Psomas, Director of Tunnel Structures at Cowie UK, looks at how to estimate mean crack-width for 'strain-softening' SFRCs which are usually adopted in tunnel- and shaft-lining applications

STEEL FIBRE-REINFORCED CONCRETE (SFRC) for tunnel linings has been used in the UK over the past 25 years. One of the main design benefits over steel rebar is the elimination of the long-term corrosion risk, resulting in enhanced durability. However, there is no established method of design analysis under the Service Limit State format that has ever been incorporated into Design Standards, and the current design rules for SFRC tunnel linings are absent in the current EC2 structural design codes, even though Model Code 2010 (fib, 2013) and the future EC2 include procedures that can be adopted for the design of tunnel linings.

This article will focus on estimating the mean crack width for 'strain-softening' SFRCs, which are usually adopted in tunnel (and shaft) lining applications. This discussion does not cover any high-performance fibre-reinforced cement composites nor tailored-made strain-hardening cementitious materials.

PROBLEM DEFINITION WITH SFRC DESIGN

In view of the extensive use of SFRC, the scarcity of reported research on tunnel applications is remarkable. Although the ultimate limit state (ULS) is covered to a significant extent in the existing norms and guidelines, the verification of service limit state (SLS) is a challenge for SFRC linings as there is no direct calculation to determine maximum crack widths.

It is often assumed that the behaviour of a structure can be approximated by the behaviour of a test beam or slab. This assumption is markedly restrictive for SFRC tunnel linings and leads to a significant underestimation of the actual resistance of the lining structure – which is an indeterminate structure – as opposed to the resistance of a beam. In fact, a linear elastic approach cannot properly take into account the beneficial effects of fibre reinforcement which become effective only after cracking of the concrete matrix when SFRC behaviour is significantly non-linear. Fibre reinforcement is known for improving crack-propagation resistance, ductility, as well as promoting multiple crack development and therefore reducing concrete permeability. This is particularly significant for tunnels where linings are potentially exposed to moderate tension and bending.

SPECIAL FEATURES OF SFRC

SFRC is a composite material so its mechanical performance in real applications displays special features which should be considered:

a. Steel fibre type

For crack-width control, the greater the number of a given type of fibres/m^3, the better the results. However, it is primarily about the right fibre rather than just the number. Thus, aspect ratio over 65 is favoured with a steel yield-strength of the wire to be at least 1,500MPa. For tunnel applications, the fibre volume fraction varies between 0.30% and 0.60% for high yield fibres.

b. Size (depth) effects

This is a known feature of SFRC resulting in a reduction of flexural tensile strength with increasing thickness of the test beam. The size dependency of flexural strength can be explained with non-linear fracture mechanics of strain-softening materials (TR63, 2007).

c. Fibre orientation

Orientation and distribution affect the performance of the SFRC. Fibres tend to align in a random 2D plane at right angles to the direction of casting or spraying. The risk of having a difference in distribution along the height increases with depth of the lining. MC2010 (fib, 2013) incorporates the fibre orientation factor in relation to the orientation of the principal stress. For tunnel linings, structural verification can be carried out using a factor of 1.0 considering isotropic fibre-plane orientation due to the uniform geometry.

It is important to note that usually tunnel linings are indeterminate structures with significant redundancy, able to redistribute the stresses. Any load propagation – if not local or concentrated – is controlled by the structural redundancy of the lining allowing multicracking to develop. It has been suggested (di Prisco *et al*, 2009) that the structural response in this situation is primarily governed by the average values of the material properties rather than characteristics. This is also relevant to SLS verification, if structural ductility rules apply. The enhanced resistance of the structure is recognised in MC2010 (up to a factor of 1.4) provided that the appropriate ductility is demonstrated.

SFRC TUNNEL LINING APPLICATIONS

Design information and requirements

Currently in the UK, where there is significant experience in precast segmental and sprayed concrete linings, SFRC accounts for a significant amount of tunnel linings. Relatively recently, applications extended to cast-in-place secondary linings, covering a wide range of design situations, including transient direct tensile loadings. Residual tensile strength requirements vary from 2MPa (low-end sprayed concrete) up to 5MPa (precast and cast in-situ).

Requirements between different types of lining differ in terms of the magnitude of the residual strength, limited by the practicalities. For example, using limited-length fibre is for spraying, dosage and distribution pipes for cast in situ. Irrespective of the application, SFRC in tunnelling is a balance between the increased cost of steel fibres and their impact on workability, which keep the steel dosages within well-defined limits.

The specification then relies on performance that relates to structural safety and serviceability. Structural safety is verified by using the appropriate safety format and adopting adequate safety factors to account for the uncertainty of the analysis model and material variability. Serviceability however, is more related to the verification of the crack width, which inevitably, in the case of SFRC, is related to the mean strain developed.

Durability of SFRC for nominal crack widths

Durability resistance of SFRC has been investigated over the past 40 years, providing a consensus on the resistance of SFRC in moderate exposure conditions and limiting crack widths in the region of 0.20-0.40mm. However, there is debate as to whether this level of crack width is adequate in aggressive chloride exposure, especially under wet and dry cycles (XD3, XC4). The challenge of providing a durable concrete structure in excess of 100 years is evident, especially in design situations where full-depth crack widths are anticipated. Several national guidelines recommend the use of coated or stainless steel fibres in aggressive environments, even if the crack widths are limited to 0.20mm. An extensive discussion on the published data and a deterioration theory for cracked SFRC has been given by Marcos-Meson *et al* (2018).

The above limits do not account for water tightness limitations which can easily result in sometimes very strict requirements for crack widths (less than 0.10mm) to ensure compliance. Hereafter, the discussion focuses on estimating SFRC under relatively small crack-widths of 0.20-0.30mm, in moderate exposure conditions under combined moderate bending and low axial thrust. In these situations, SFRC is considered to have superior durability to bar-reinforced structures due to the better quality of cold-drawn wire steel and improved interface between fibre and concrete matrix.

Often in tunnelling, the durability of SFRC concrete depends upon the mean crack width and the ability to self heal, preventing water ingress. Crack widths less than 0.3mm are specified as a target upper crack-width limit for SFRC.

Design approach and requirements for crack control

It is not possible to calculate crack widths in statically indeterminate structures without performing a non-linear analysis. If conventional numerical FEA is performed, the mean strain can be calculated along a characteristic length (‘l_{cs}’). for which the upper limit can be taken as section depth h, according to MC2010. If non-linear analysis is considered, then if the tensile strain exceeds the elastic limit, the lining will crack. The number of cracks and the crack widths will depend upon the mean tensile strain, provided that more than one crack is formed. Multicracking is necessary for crack width control but not sufficient for achieving the prescribed maximum crack width in strain softening materials. Thus, it is more appropriate to refer to ’mean crack widths’ when addressing SFRC crack widths. Provided that ULS is verified (in terms of stress limits) and cracks are controlled, it is likely there is no admissible mechanism of failure; the verification of the design is governed by the crack width requirements.

SFRC characterisation

In order to derive the material properties used in the current structural concrete design methods, tests yielding a bending load deflection curve are required. From these, the flexural strength can be translated into direct tensile strength. Among the plethora of various test methods, the three-point notched beam test (BS EN 14651) has been now established in Europe as the most representative test for SFRC characterisation and derivation of structural design input parameters. The purpose of a notched test is to fix the location of the fracture plane in a strain-softening material. From this test, ductility and strength ratios can be derived. MC2010 requires the ductility ratio f_{R3k}/ f_{R1k} and strength ratio f_{R1k} / f_{Lk} to be greater than 0.5 and 0.4 respectively, as a minimum. In practice, to ensure crack control (multicracking) in bending, the minimum ductility ratio needs to be at least 0.9 or greater.

The strength indices are calculated from the tests and relevant for SLS is f_{R1}, which defines the tensile strength for crack widths up to 0.5mm, by $f_t = 0.45\, f_{R1}$.

The strength indices in Figure 1 can be used to derive the design strength of the lining, although certain adjustments should be carried out to account for the size effects. It is important to note that the small-scale bending test can correlate with the residual strength derived from 'full-depth' scale tests as shown below.

Design assisted by testing

Large hydraulic tunnels in urban areas are required to sustain tensile strains due to internal surge pressure. The verification of SFRC required full 'thickness' structural testing to be performed. This type of testing ensured the ductility of SFRC at full thickness (deflection-hardening), a sound representation of fibre distribution and the elimination of scale effects. For a particular project in London, large-scale four-point bending testing was carried out at the Building Research Establishment (BRE) at Watford, UK, and small-scale three-point tests were also performed and correlated to the large-scale testing. This example is described by Eyre *et al* (2015), where in order to accurately measure crack widths and strains developing in concrete and steel fibre-reinforced concrete beams under load, digital photography and particle image velocimetry (PIV) and fibre-optic Bragg gauge techniques were used. The aim, of that testing programme was to derive the load-deflection response and then estimate the development of crack widths versus time.

From the large-scale tests, the mean flexural tensile strength was compared to f_{R1} (small-scale test) as the strength related to SLS; it provided a good fit to the test M vs ε_{av} curve *(see Figure 4 below)*. The results – given later – adopted the factor k=0.88, which relates to size effects and is explained elsewhere (Johnson *et al*, 2017). So, for this example, using the figures from Figure 1 would yield:

$$ft = 0.45\, f_{R1k}\, k = 0.45 * 6.1 * 0.88 \text{ MPa} = 2.4 \text{ MPa} \quad (1)$$

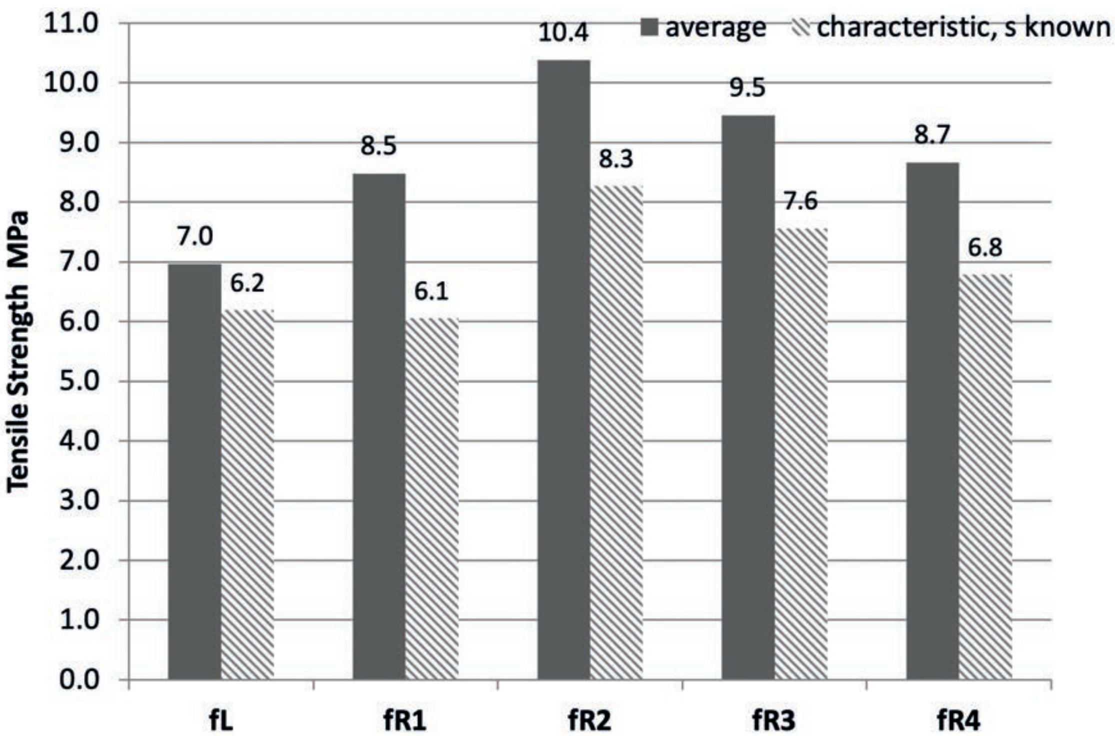
The large-scale tests produced a constant bending moment area (Eyre *et al*, 2015). The material exhibited deflection-hardening behaviour with the mean first crack tensile stress $f_{t,e}$ reaching 5.73 MPa, retained for significant strain level. The corresponding mean cracking strain $\varepsilon_{av.} = 173\mu\varepsilon$.

The secant elastic modulus can be calculated as:

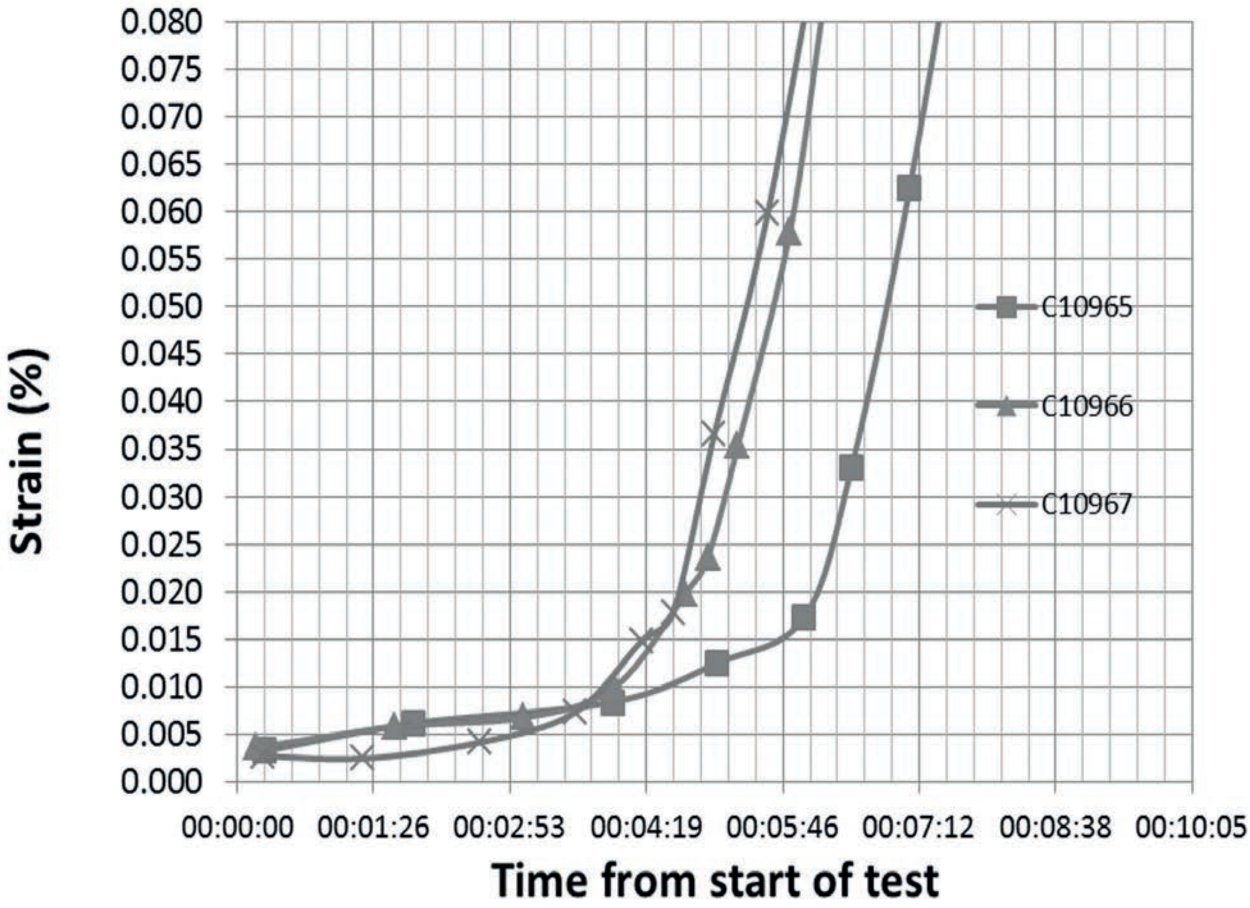
$$E = f_{t,e} / \varepsilon_{av.} = 5.73 \text{ MPa}/0.173e\text{-}2 = 33 \text{ GPa} \quad (2)$$

The mean tensile strength (variation in large test was less than 5%) is:

$$f_{ctd,s} = 0.45\, f_{t,e}\, k / \gamma_{m,sls} = 0.45*5.73*0.88 / 1.00 = 2.3 \text{ MPa} \quad (3)$$



Above: Figure 1, Strength indices to EN14651 – hydraulic tunnel lining in London



Above: ***Figure 2, Strain development with time from the full-scale tests***

Then ULS design strength assuming a material factor of $\gamma_{m,uls}$ = 1.50 is:

$$f_{ctd,u} = f_{ctd,s} / \gamma_{m,uls} = 1.5 \text{ MPa} \quad (4)$$

The design analysis of the lining was facilitated by carrying out non-linear FE analysis. The purpose of this was to capture in the analysis the residual tensile capacity of SFRC and to derive the critical mean tensile strain. The stress-strain curve of the SFRC material was adopted from the large-scale tests. The FEA methodology and the numerical modelling is not part of this paper as the non-linear analysis is the state of practice for SFRC. The important aspect to note is that non-linear FEA enabled the derivation of critical cracking strain after the stress redistribution. This is a necessary stage in order to determine the critical strain required for the crack-width estimation.

The design 'assisted-by-testing' methodology can be then summarised as follows:

1. Derive load deflection vs time, and load vs deformation curves from testing;
2. Deduce from data post-processing analyses the total tensile strains;
3. Determine, from the FEA analysis of the lining, the critical strain distribution;
4. Produce the strain versus time graph and identify the critical strain limits *(figure 2)*;
5. Calculate the strains and estimate the average crack widths (in this case PIV and FOBG).

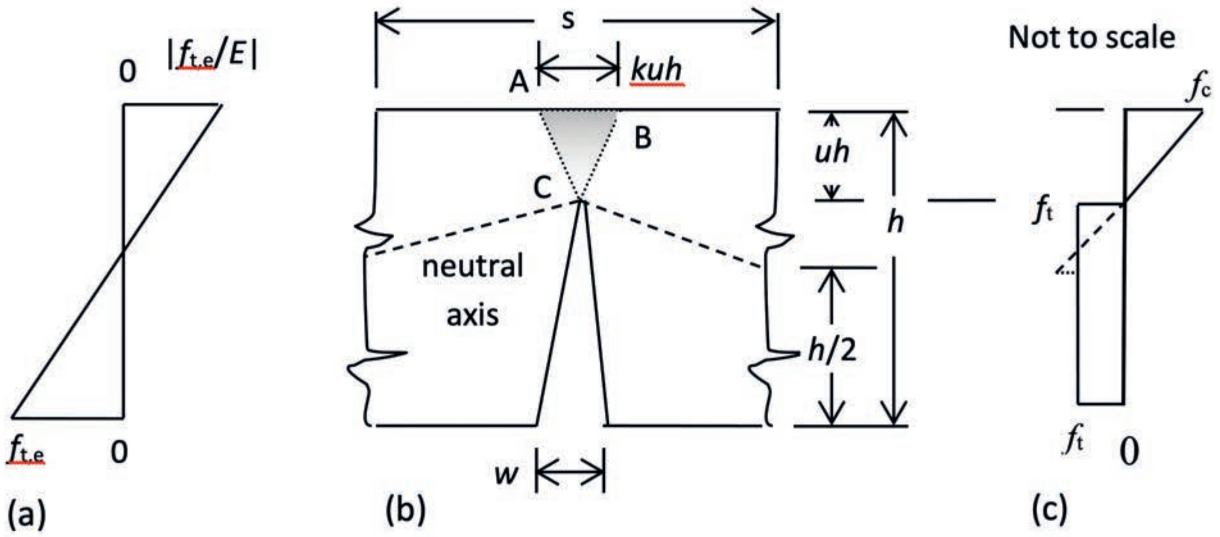
The large-scale tests assisted in designing the tunnel lining for SLS. This design methodology enables the development of an analytical design approach for SLS verification *(see below)*, which can be used for future applications.

SERVICE LIMIT STATE VALIDATION

A relatively simple model was developed to validate the SLS design shown in Figure 3 - a length 's' of a member of rectangular cross-section of unit breadth and depth h, subjected to bending moment M. There is a single crack, of width w, shown at the bottom surface. The assumed crack spacing is 's'. At distance s/2 from the crack, the behaviour is elastic, with stresses as in Figure 3 (a), with:

$$6\,M = f_{t,e} = h^2 = E\,h^2\,\varepsilon_{av}\,, \text{(for b = lm)} \quad (5)$$

For SLS verification, crack width is always less than 0.5 mm. If the 'linear model' of the Model Code (*fib*, 2013) is used, it gives the stresses in figure. 3(c), with f_t from equation (1). Here, for prediction of test results, f_t is based on the mean value of f_{R1}. Its characteristic value is used in design. For simplicity, the tensile region of the uncracked concrete up to its cracking stress (the dashed lines) has been ignored as this complicates the analysis and barely affects the results. The objective is to obtain curves of



Above: Figure 3, (a) Elastic strains midway between cracks (b) Elevation of length 's' (c) Elastic-plastic stress distribution at a crack

bending moment in terms of mean tensile strain, ε_{av}, for several assumed numbers of cracks (based on the full scale tests in the almost constant-moment region of length 1.645m); that is, for given ratios s/h, where s is the crack spacing. This is done for one, three and six cracks, so the ratios s/h are 5.48, 1.83 and 0.91. For fully-developed cracking, crack spacing can be taken as the crack depth, which will be less than 0.91 h. This is close to the theoretical ratio proposed by MC2010.

The key assumption of this 'hinge' theory is that the maximum compressive stress is constant at ft over some length [k u h], where k is to be determined, and u (= x/h) is the non-dimensional depth in compression at the crack. The compressive stress falls to $|f_{t,e}|$ at the ends of length s.

The extension of length CD in Figure 3(b) is:

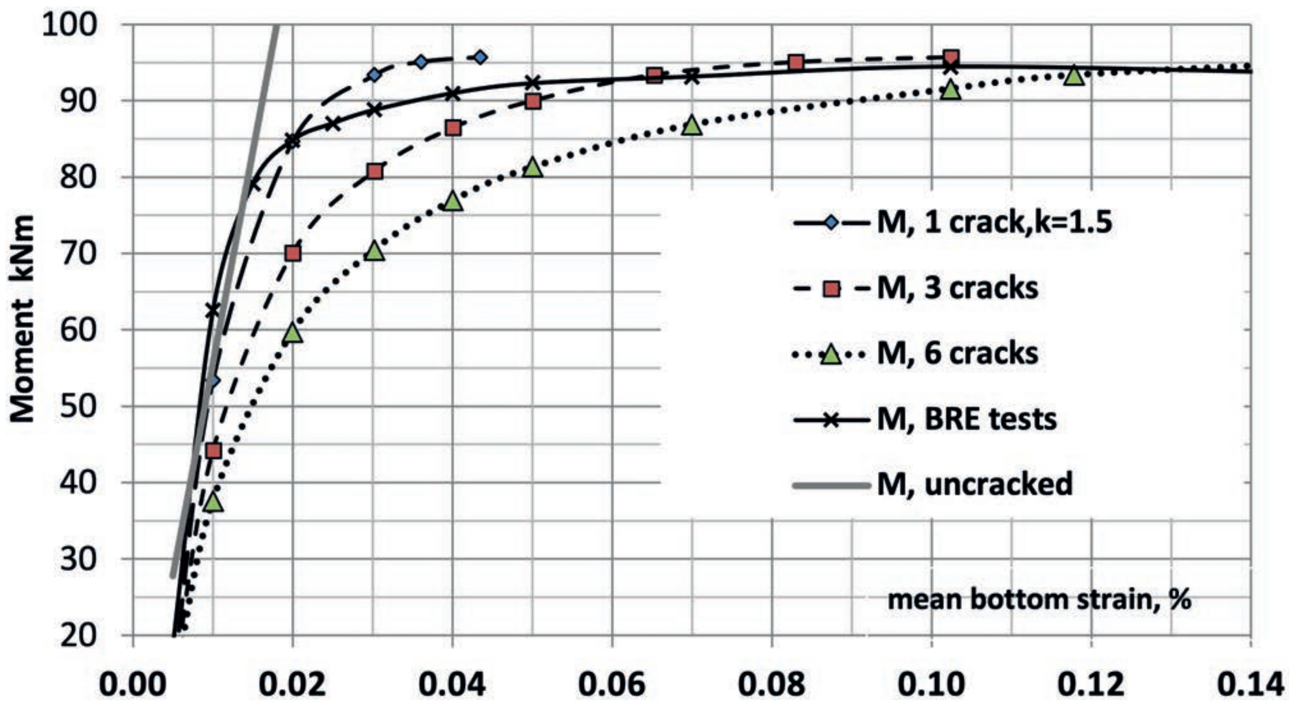
$$k\,u\,h\,f_c\,/\,E, \tag{6}$$

so the hinge rotation is

$$(k\,u\,h\,f_c)\,/\,(E\,u\,h) \tag{7}$$

It is also the angle between the faces of the crack,

$$w\,/\,[h\,(l-U)]. \tag{8}$$



Below: Figure 4, Mean test results, and predicted bending moments as a function of mean tensile strain

Equating these rotations gives:

$$w = k\,h\,(l - u)\,f_c / E \qquad (9)$$

The steel fibres are closely spaced, so at the face of the crack, the stress ft is transferred to the concrete within a short distance of the crack. The surface tensile strain is assumed to be f_t/E at the crack and $f_{t,e}$ at the ends of length s. These strains are similar (in contrast with f_c/E), so a linear variation can be assumed between them, over length ($s - w/2$).

Assuming that crack width w is much smaller than s, and using equation (5), the mean tensile strain over length s is given by (no axial force considered N=0):

$$\varepsilon_{av} = (w/s) + [(6\,M / h^2 + f_t) / 2\,E] \qquad (10)$$

For longitudinal equilibrium at the crack,

$$f_c u\,h / 2 = ft\,h\,(l - u) \qquad (11)$$

At the crack, the bending moment is given by

$$12\,M / h^2 = f_c u\,(3 - 2\,u) + 6\,f_t u\,(l - u) \qquad (12)$$

Elimination of w, M and f_c from eqs (9) to (12) leads to

$$u^3 + 2\,u^2 - 4\,K_1\,(l - u)^2 + 2\,K_2\,\varepsilon_{av}\,u - 4\,u = 0 \qquad (13)$$

where

$$K_1 = k\,h / s \qquad (14)$$
$$K_2 = E / f_t \qquad (15)$$

This is solved for assumed values for ε_{av}; then w, M and f_t are found. This gives $M(\varepsilon_{av})$ for any assumed number of cracks (s/h) and ratio k.

The best match with the curve from the tests was found with k = 1.5. Elimination of u and f_c from equations (9), (11) and (12) gives a curve $M(w)$ which is independent of the crack spacing, as it is based only on the model at the crack.

The mean curvature φ_{av} can be found by assuming that the depth of the member in tension, d, varies linearly from h (1 – u) at the crack to $h/2$ at the ends of length s. Hence,

$$d_{av} = h\,(3 - 2\,u)/4 \qquad (16)$$

and from Eq. (10):

$$\varphi_{av} = \varepsilon_{av}/d_{av} \qquad (17)$$

The rotation ϑ of length s is then:

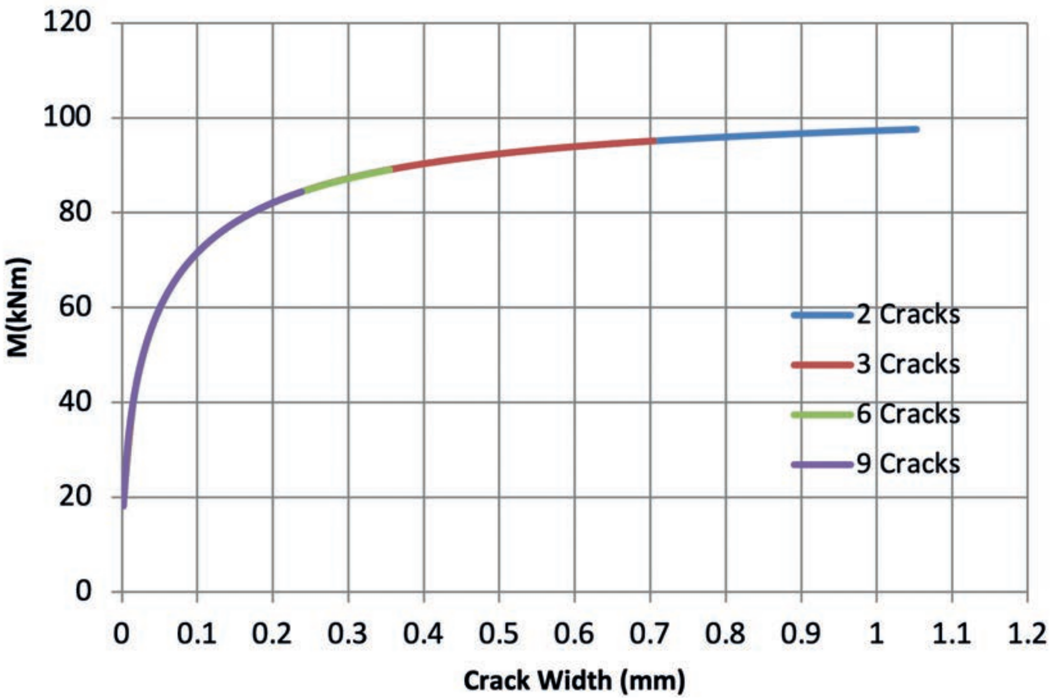
$$\vartheta = \varphi_{av}\,s + w/[h\,(l - u)] \qquad (18)$$

The above theory has been tested producing Figure 4:

The curves shown in Figure 4 (Johnson, 2014) were calculated for a 1m-wide beam using these values: h = 0.300 m, f_t =1.5 MPa, E = 33 GPa, k = 1.5 and ratio of crack spacing s to thickness h as 5.48, 1.83 and 0.914. Interpolation between them can be used. The 'uncracked' line in figure 4 is based on the secant modulus at 'first cracking', that is, when a crack becomes wide enough to be noticed. The predicted curve, 'one crack', meets the test curve at ε_{av} = 0.02%, or 200με, slightly above the expected cracking strain for normal concrete of this strength, but very close to the mean (173με) observed in BRE tests.

In this example, the curve of Figure 5 enables crack widths to be predicted based on the total strain. It shows that the bending moment continues to increase until the mean width reaches about 0.5mm – an important result because up to the serviceability limit, likely to be 0.20mm or less, behaviour is

Below: Figure 5, Predicted bending moment as a function of mean crack width, for any number of cracks



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Notation (based on EN 1992-1.1 and Model Code 2010)

- E effective Young's modulus for concrete in compression
- L length of region of almost constant bending moment
- M bending moment in a member of unit width at a cracked cross-section
- b breadth of member, taken as unity and omitted from most equations
- f_c maximum compressive stress in the concrete at a cracked cross-section
- f_{cy} compressive 'yield' strength of concrete
- f_{Lk} first crack 'characteristic' stress calculated from BS EN14651 test
- f_{R1k} 'characteristic' stress calculated from BS EN14651 test, at crack mouth opening of 0.5mm
- f_{R3k} 'characteristic' stress calculated from BS EN14651 test at crack mouth opening of 2.5mm
- ft assumed flexural tensile stress across a crack at the neutral axis
- $f_{t,e}$ extreme fibre stress for an elastic uncracked cross-section
- h depth of a member or test specimen
- k hinge length factor
- s length of beam affected by a crack, and assumed final crack spacing
- u non-dimensional ratio x/h
- w width of a crack at the surface in tension
- x depth of compression zone at a crack
- ε_{av} mean strain at the surface in tension over length L
- φ_{av} mean curvature of length L of the beam

clearly deflection-hardening and therefore stable in this strain range. The analysis above can be extended to include circumferential tension or compression, and for crack widths up to 2.5mm (typically ULS limit). The stress fc reaches the assumed 'yield' value of 25MPa, when the bending moment reaches 91.3kNm, after which yielding has been allowed for.

Yielding begins to affect the curves shown, but it is still small at 0.5 mm. It is interesting to note that one 0.5mm crack in a length of 270mm increases its mean curvature to 3.7 km^{-1}. In a tunnel lining, this great increase would cause redistribution away from the hinge region.

Further expansion of this approach can be achieved by introducing the axial force N and limiting the strain to the chosen critical value (0.03% for this application based on FEA). Notwithstanding this, it is encouraging that the methodology above can be used to predict the SFRC response for an assumed number of cracks.

This model, using mean strains and uniform widths of uniformly-spaced cracks, cannot represent the complexity and variability of cracking, however, it predicts their average behaviour.

These findings are based on a single series of tests, and one type of fibre ($40kg/m^3$, Bekaert 5D 65/60-). Further analysis on the statistical distributions of crack width or spacing was not undertaken.

CONCLUSION

Tunnel linings can be designed and verified as steel fibre-only reinforced structures.

The design verification for the SLS requires testing, so that the estimation of the average structural crack is feasible.

The small-scale three-point beam tests can be correlated with full-depth scale provided that ductility ratio of unity (or above) is achieved.

It is suggested here that high ductility is required to achieve meaningful crack-width control. Additionally, a simple analytical model is presented to validate the large-scale tests and attempts to predict behaviour.

The model presented here fits the test data well, but needs to be tried out in tests for a variety of concrete mixes and fibres.

Thus, this work also exposes the need for stronger evidence from tests and site measurements so as to streamline the design of SFRC tunnel linings and to reduce uncertainty.