SFRC – SHEAR LOAD BEARING CAPACITY AND TUNNEL LININGS
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Abstract
Recently, the use of structural elements made of steel fibre reinforced concrete is becoming more common. Steel fibres are used instead of ordinary steel reinforcement or in addition with reinforcing bars. Both, under service and ultimate loading conditions, the fibre reinforcement is subjected to carry tensile loads. The addition of steel fibres to plain concrete changes its mechanical properties. Depending on the type and amount of fibres an increase in ductility and a better cracking behaviour can be achieved. Especially by the use of steel wire fibres remarkable stresses can be transferred across cracks. The fibre itself can be seen as a kind of reinforcement. Verification concepts for SFRC structural members can be derived from this principle.
Based on these considerations, research work carried out with respect to the shear design of SFRC and SFsRC beams and tunnel linings will be presented in this paper.

1. Steel Fibre Concrete – Shear Load Bearing Capacity

The investigations concerning the shear load-carrying behaviour of steel fibre reinforced beams carried out within the scope of a Brite/Euram programme showed that the empirical design approach according to Rilem TC 162 TDF 72/ results in a safe estimation of the shear load-carrying capacity (test beams see Fig. 1.1)
Based on further tests carried out at the iBMB of the Technical University of Braunschweig and on data taken from an intensive literature /3/ study, the steel fibre action on the different components influencing the shear load-carrying capacity of rectangular beams became further investigated. It could be established that the main effect of the steel fibres can be seen in the tensile force component acting in the inclined crack.
The steel fibre orientation plays an important role for the effectiveness degree of the steel fibres, as it could be shown that the post cracking tensile strength depends to a high degree on the concreting direction and loading direction, respectively (/4/, /5/, /6/). This
effect, due to the inclined crack, plays a fundamental role for the shear load-bearing capacity /3/.

\[ F_u = 240 \text{ kN} \]

VK 1.2/3, without stirrups
Steel fibre content 40 kg/m³

VK 1.3/1, without steel fibres
\[ r_w = 0.07 \% \text{ (stir. Å 4 mm, cc= 18 cm)} \]

VK 1.3/3
Steel fibre content 40 kg/m³  \[ F_u = 356 \text{ kN} \]

VK 1.4/1
Steel fibre content 0 kg/m³  \[ F_u = 340 \text{ kN} \]

Fig 1.1: Failure modes of different test specimens with the same longitudinal reinforcement ratio, different steel fibre contents and different shear reinforcement ratios /1/

One main observation derived from the test was, that the crack propagation for beams without stirrups, but with steel fibres, takes place much more moderately. These cracks reach for more into the upper beam region than it would be the case for similar beams without steel fibres. On the other hand side, the single phases of the crack propagation could not be determined as exactly as it is the case for beams without steel fibres /3/. For the phases off crack development see Fig. 1.2.

Fig 1.2: Crack propagation for beams without and with steel fibres /7/
Another important role plays the increase of the dowel action of the longitudinal reinforcement caused by the steel fibres. This results in a stronger redistribution of the shear force from the compression zone towards the dowel action of the longitudinal reinforcement.

Based on the experimental and theoretical investigations a mechanical model for the shear carrying behaviour could be developed by Rosenbusch /3/, which allows for a continuous transition between beams with and without shear reinforcement. The steel fibre action can mainly be seen in the additional carrying of tensile forces within the cross section of the beam. An influence of the steel fibres on the height of the compression zone, friction between cracks, shear slenderness and dimension effect can, based on the results of these investigations, be neglected. The steel fibre action can be seen equivalent to the action of normal reinforcement, if the fibre orientation is regarded. Therefore, the design model can be used not only be used for beams, where only the required minimum reinforcement according to DIN 1045-1 is necessary, which can be replaced by steel fibres, but also for beams with a combined reinforcement of re-bars and steel fibres.

The design model is based on the well-known truss-model, with the tensile struts consisting of normal stirrup reinforcement and an additional tensile force component due to the steel fibre action.

A comparison between experimental failure loads from approximately 130 test specimens with and without steel fibres with the calculated failure loads according to Rosenbusch’s model and according to the German Concrete Code DIN 1045-1 is given in Fig. 1.3. The comparison in Fig. 1.3 is based on beams with a rectangular cross-sections and a shear slenderness a/d ≥ 3.0. For the verification of beams with steel fibres according to DIN 1045-1 an equivalent shear reinforcement ratio according to /3/ was used.

Fig. 1.3: Comparison between Rosenbusch’s approach with DIN 1045-1 (see /3/)

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Based on existing safety concepts /8/ a design proposal for steel fibre reinforced beams was developed. Fig. 1.4 shows as an example a design diagram according to Rosenbusch. These design diagrams enable a simple dimensioning of steel fibre reinforced beams, i.e. the determination of the necessary post cracking steel fibre tensile strength for beams with a given shear design load $V_{ed}$, longitudinal reinforcement ratio $r_L$ and a web reinforcement ratio $r_w$.

\[
V_{Ed} = k_w \cdot d \cdot \left( f_{R,4,d} \right) \quad \text{[MN/m²]}
\]

\[
N_t \cdot f_{R,4,d} \quad \text{[MN/m²]}
\]

Example: BST 500 C 35/45 $d = 45$ cm

![Design Diagram]

Fig. 1.4: Dimensioning diagram acc. to Rosenbusch /3/ for beams with a rectangular cross-section

2. Steel Fibre Concrete – New Development for Tunnel Linings

The use of steel fibre reinforcement, combined with conventional reinforcement, can be very advantageous for the construction of tunnel linings. Tunnel linings have to be designed for a serviceability (SLS) and ultimate limit state (ULS). In addition to the optimisation of reinforcement, the tunnel lining has to be designed as thin as possible, in order to reduce its bending resistance. The stiffness relation between soil and lining influences the bending moments in the lining considerably. Therefore, a reduced bending resistance of the lining leads to reduced bending moments and a growing membrane effect. This is advantageous, as a membrane effect results in a better utilisation of the lining material. The increased deformation capability of thinner tunnel linings induce a greater support reaction of the soil. In this context a combination of normal reinforcement with steel fibre reinforcement can be useful and leads to better results.
The investigations carried out in this context refer to inner tunnel linings of cast in-situ concrete and prefabricated segments. In order to clarify the load carrying and cracking behaviour of these tunnel linings under real loading conditions, extensive number of tests on single span and continuous girders, as well as full scale tunnel linings were carried out with different reinforcement concepts (Plain concrete, reinforced concrete (RC) and steel fibre strengthened reinforced concrete (SFsRC)). The test results concerning the load carrying and cracking behaviour were used to calibrate the additional FE-calculations for the overall load-carrying and deformation behaviour of tunnel linings in interaction with the surrounding soil. Figure 2.1 shows a tunnel segment in the test stand.

![Tunnel segment in the test stand](image)

Fig. 2.1: Tunnel segment in the test stand

The calculation and design of tunnels is based in Germany on different codes an design concepts. For the serviceability limit state (SLS) of a tunnel, the watertightness according to /9/ is normally given if either a minimum height of the compression zone x a minimum crack width can be proven. Furthermore the allowable steel and concrete stresses according to DIN 1045-1 /10/ for the ultimate limit state (ULS) have to be kept. The investigations summarised here were carried out to provide information about which limit state is decisive for the design of tunnel linings.

The crack width model for steel fibre concrete according to /11/ is only valid for single crack formation and bending. For combined compression forces N and bending moment M one has to expect the formation of multiple cracks. The corresponding computational models are still missing. Niemann /12/ has developed crack with models for steel fibre strengthened reinforced concrete (SFsRC) under load and restraint, but models for tunnel specific border conditions had so far not been published and therefore one of the aim of this research project was to either verify the existing models, to adapt them, or to develop a new model for the specific border conditions.
The other aim was to show which design state (ULS or SLS) is decisive for the tunnel design and which reinforcement concept is able to fulfill all given border conditions in the best possible way.

2.1 Steel fibre concrete – crack width model

The crack width model developed for SFsRC here is based on the assumptions of Model Code 90, incorporates the work of Niemann /12/ and has been modified in various aspects and considers the special aspects for the tunnel design, such as high concrete cover of the reinforcement, compressive reinforcement and simplified iterations. Fig. 2.2 gives the general design assumptions.

![Crack width model](image)

**Finished cracking**

\[ S_{sfr} = S_{fr} + \frac{f_{ctm}}{E} \]

**Introduction length**

\[ L_{int} = \frac{f_{ctm}}{E} \frac{d}{r_{eff}} \]

\[ a_0 = \frac{f_{ctm}}{E} \]

\[ b_0 = \frac{f_{ctm}}{E} \frac{r}{d} \text{ (acc. to MC 90: 1.80 for ARB)} \]

\[ a_1 = \frac{t_{eff} + t_{cm}}{t_{eff} - t_{cm}} \text{ (t}_{eff} \text{: improved bond by steel fibres} \]

\[ r_{eff} = A_{fr} \left( h - x \right)^{0.5} \text{ with } \left( h - x \right)^{2} < 2.5 \cdot d \]

**Strain difference**

\[ (\varepsilon_{fr} - \varepsilon_{cm}) = a_0 \cdot b - a_1 \]

\[ \varepsilon_{fr} = \frac{\Delta L - \frac{1}{2} a_{eff} \cdot b}{A_{fr}} \]

\[ \Delta L = \frac{t_{eff}}{E_{fr}} \left( 1 + a_0 \cdot r_{eff} \right) \]

\[ a_{fr} = \frac{f_{ctm}}{E_{fr}} \text{ (considers the reduction by fibres)} \]

\[ s_{fr} = \frac{1}{A_{fr}} \left( \frac{d}{E_{fr}} \frac{M_{sfr}}{z} + \frac{1}{A_{fr}} \frac{d}{E_{fr}} \frac{M_{fr}}{z} \cdot \left( h - x \right) \right) \cdot \frac{1}{0.85 \cdot d} \]

\[ N = \text{negative as compression force} \]

\[ h = (1 - a) \cdot b + (a_{fr} \cdot r_{eff}) + (a_{fr} + 1 - a) \cdot b \]

\[ b = 0.6 \text{ short time action; } b = 0.4 \text{ long time action} \]

\[ r_{eff} = A_{fr} \left( h - x \right)^{0.5} \text{ with } \left( h - x \right)^{2} < 2.5 \cdot d \]

**Crack width, crack distance**

**Max. crack distance**

\[ s_{max} = 2 L_{int} \quad \left( s_{max} \geq 2 s_{min} \right) \]

**Max. crack width**

\[ w_{max} = S_{max} \left( \varepsilon_{fr} - \varepsilon_{cm} - \varepsilon_{fr} \right) \quad \text{Assumption: } \varepsilon_{fr} = 0 \text{ (shrinkage)} \]

Fig. 2.2: Crack with design assumptions
Caused by the steel fibres (considered here by the effective post cracking tensile strength value \( f_f \), including all influences due to constant loading, height and loading combination) the introduction length \( L_{ci} \) is, compared to normal reinforced concrete, reduced. Furthermore the strain difference \( (\varepsilon_{sm} - \varepsilon_{cm}) \) between steel and concrete is smaller than for normal concrete. Both effects lead to a crack width reduction for building members with a combined re-bar and steel fibre reinforcement.

A reduced relative eccentricity \( e/h = M/(N*h) \), as well a possible improved bond behaviour caused by the steel fibres contribute to this effect. Nevertheless, if these design model was applied to the measured crack width obtained from the tests, considerable differences occurred between the measured and calculated crack with. Fig. 2.3 shows the differences. The reason for these differences have to be seen in the effective width of the reinforcement \( h_{eff} \). The effect of the reinforcement does not reach the edge of the building member, thus causing a higher crack width than expected. This effect increases with lower relative eccentricities.

![Fig. 2.3: Comparison between calculated and measured crack widths](image)

It can be seen, that with decreasing effective height \( e/h \) a reduction in the influence zone of the reinforcement \( h_{eff} \) takes place, causing a higher difference between calculated crack width \( w \) at the reinforcement and the measured crack widths \( w^* \) on the surface of the test specimen. If the model given here is modified according to Fig. 2.4, a more or less complete agreement between calculated and measured values can be achieved.

![Fig. 2.4: Modified crack width model](image)
2.2 Tunnel modelling – FE analysis

Based on the extensive studies carried out, a finite element model was developed on the basis of the ANSYS FE-programme, able to carry out tunnel lining computations for in-situ concrete tunnel linings, as well as segmented tunnel linings with the consideration of the load bearing action of tunnel joint in longitudinal and ring direction.

The modelling of the re-bar reinforcement was carried out discretely, connected to the concrete elements by discrete springs, thus modelling the bond characteristics between steel and concrete. The stiffness relations obtained from the tests were as well incorporated into this model. This means, that this model could be used for the computational verification of the tests carried out. These control computations showed an excellent agreement with the test results, with meant that this model was suitable for computer studies in order to determine the optimal tunnel lining, able to fulfil all set requirement, i.e. the SLS and the ULS.

Figure 2.5 shows the model developed for the FE-analysis, the following sensitivity studies and safety considerations.

Fig. 2.5: FE-model for the interaction tunnel-soil

In order to find out which parameter significantly influence the limit states of the tunnel lining design, a sensitivity study, based on this model, was carried out. At first the basic variables and their statistical distributions had to be defined (normally of log.-normally distributed). The following procedure for this study was chosen:
FEM and Monte Carlo simulation (MCS) with the chosen basic variables gives the failure probability for the defined border conditions as well as the statistical significance of the basic variables. Application of a multiple regression (linear) on the system results in order to formulate a limit state equation depending on the basic variables. Derivation of sensitivity factors and/or partial safety factors.

In order to control this procedure, simplified limit state equations have been mechanically developed by Hemmy /13/ and were statistically evaluated according to the first order reliability method (FORM). Figure 2.6 shows the aim of this investigation, as well as the basic variables and the border conditions used.

**Physically nonlinear section forces**

Contradiction: Section forces with mean values & dimensioning with characteristic values

- Building construction: DIN 1045 -1, ULS: special design concept (strengths, partial safety factors)
- Tunnel construction: strengths ?, partial safety factors ?, SLS ?

Which variables significantly influence the serviceability limit state?

**Reliability based FE-analysis (Monte Carlo Simulation)**

![Diagram showing limit state conditions and basic variables](image)

Fig. 2.6 Motivation and border conditions of the FE-analysis

These investigations showed, that the two soil parameters lateral pressure coefficient \( K_0 \) and stiffness module \( E_s \) have a significant influence on the watertightness of the tunnel lining, whereas the material properties are hardly significant. Only a slight correlation between the uncracked height \( x' \), the material basic variables and the soil weight can be observed. The strongest correlation was obtained for the lateral pressure coefficient (see Fig. 2.7).

Based on these investigations it could be shown that the SLS is decisive for the tunnel lining design. From the limit state equation, partial safety factors \( \gamma \) and sensitivity factors \( \delta \) can be developed according to Schneider et al. /14/ for a given safety index \( \beta \) (see Fig. 2.8). As a result of these studies it is recommended to carry out the SLS design with the mean values and to apply an overall safety factor for the system load bearing capacity \( G_{sys} \) of 1.15 for ductile failure and a factor \( G_{sys} \) of 1.30 for brittle failure.
From multiple regression:

\[ g_k = (680.28 \cdot K_0 + 0.94 \cdot E_k - 214.15) - x_{lim} \]

\[ x_{lim} = 100 \text{ mm} \]

Fig. 2.7: Significant basic variables, limit state condition and correlation for a non-reinforced tunnel lining.

Fig. 2.8: Determination of partial safety factors based on FORM

\[ g_k = \frac{1 + k_0 \cdot V_k}{1 - \phi_k} \cdot V_k \]

Safety index

\[ b = \frac{\phi_0}{\theta_k} \cdot P_f = F(b) \]

Sensitivity

\[ a_i = s_i / s_0 \cdot a_i \]

Partial safety factors

Lateral pressure coef. \( K_0 \)

Stiffness module \( E_k \)

Materials \( g_k = 1.0 \ldots 1.1 \)

Partial safety factor recommendation

\[ g_{sys} = (\chi_0 / \chi_k) \cdot f_r \text{ for SLS} \]

Mean values for action / resistance
2.3 Deterministic parameter studies

In order to obtain information about the effectiveness of the different kind of possible reinforcements, additional parameter studies were carried out, see Fig. 2.9.

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<th>Computation results:</th>
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<td>- t = 100</td>
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![Graph showing computation results](image)

Fig. 2.9: Investigated parameters and result examples

It can be seen from Fig. 2.9 that the best results were obtained for a steel fibre strengthened reinforced concrete (SFsRC) with a reduced wall thickness of 20 % (i.e. 240 mm instead of 300 mm). This tunnel lining showed an uncracked height x’ of 174 mm and the best rotational capacity Q, which means that this tunnel lining is superior to a conventionally reinforced tunnel lining with a height of 300 mm.

The greater deformability of such a tunnel lining can be seen in Fig. 2.10. This figure shows a comparison between a conventionally reinforced tunnel lining and a SFsRC tunnel lining. The FE study shows that the cracks in the SFsRC tunnel lining extend over a wider area, which indicates the higher deformability of such a tunnel lining. On the other hand, the cracks do not penetrate as deep in the cross section, as can be seen for the conventionally reinforced tunnel lining. It could be shown, that such a tunnel lining fulfils all requirement with regard to the SLS and ULS, even for “soft” soil conditions.
In-situ concrete lining

\[ R = 3 \text{ m}, \ h = 10 \text{ m} \]

Reinforced concrete

\[ h = 30 \text{ cm} \]

Steel fibre-strengthened reinforced concrete

\[ h = 24 \text{ cm} \]

20% higher deformations

Fig. 2.10: Computational crack zones for linings with different reinforcements

2.4 Dimensioning diagrams

In order to provide the practical user with some design aides for the SLS and ULS, dimensioning diagrams for the necessary reinforcement ratio or the necessary post cracking tensile strength \( f_{td} \) of the steel fibre concrete were developed. As an example two diagrams will be given here (see Fig. 2.11 and 2.12 next page.).

The assumptions for developing these diagrams are given in /13/ in detail. For the concrete a C30/37 was assumed, if significantly higher concrete grades are used, the diagrams for the SLS can lead to unsafe results. In such a case it is recommended to use the formulae given in /13/ as a basis for the computation.
3. Summary

The shear design concept presented here would make it possible to replace the minimum re-bar shear reinforcement which has to be provided according to the present standards.
by steel fibres. It considers the specific failure mechanism observed for steel fibre reinforced beams and/or beams with a mixed reinforcement. As far as tunnel linings are concerned, the concept presented here provides the user with improved, technically as well as economically, design rules. The combined reinforcement concept leads to reduced crack width, compared to conventionally reinforced tunnel linings, as well as to a reduction of the wall thickness. This, in combination with nonlinear design and computation methods, leads to a more robust and economical design.

References

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11. Deutscher Beton und Bautechnik-Verein e.V., „Merkblatt Stahlfaserbeton”, Oktober 2001